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PRESTRESSED CONCRETE INSTITUTE

Vol. 4 Dec. 1959 No. 3

JOURNAL

OF THE PRESTRESSED CONCRETE INSTITUTE

CONGRESS
SERIAL RECORD
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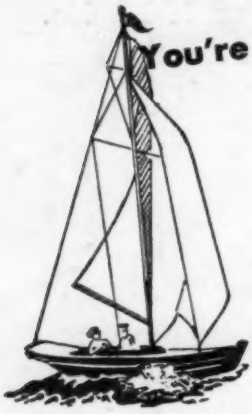

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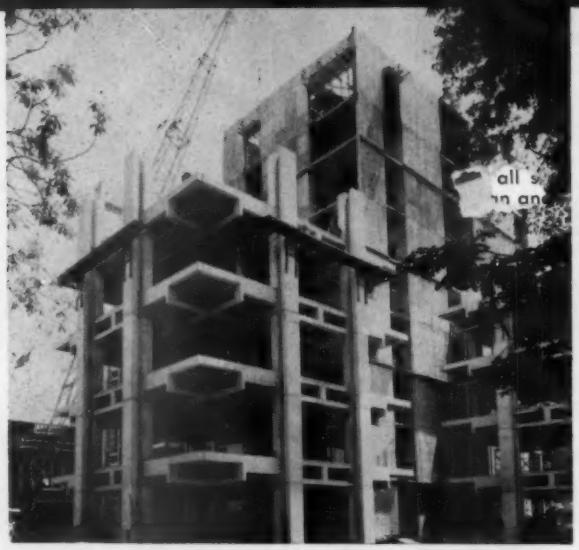

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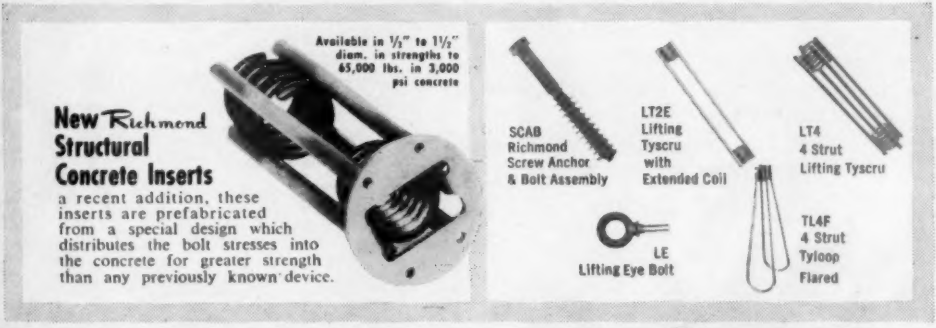
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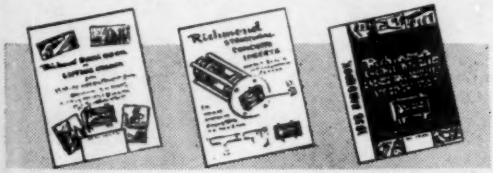

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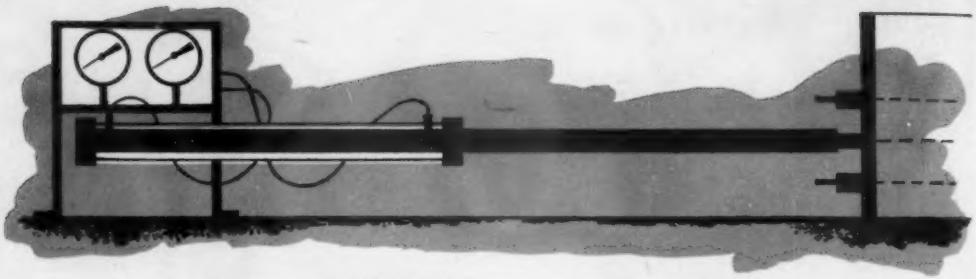

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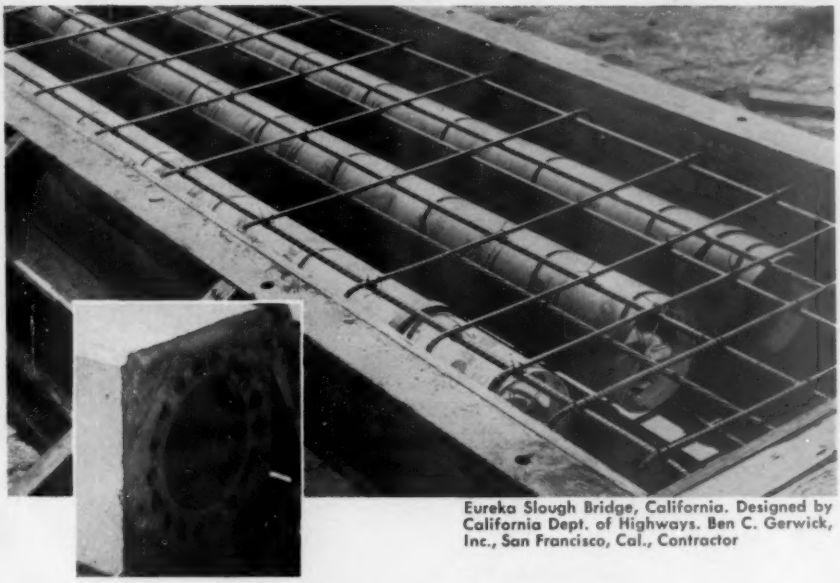

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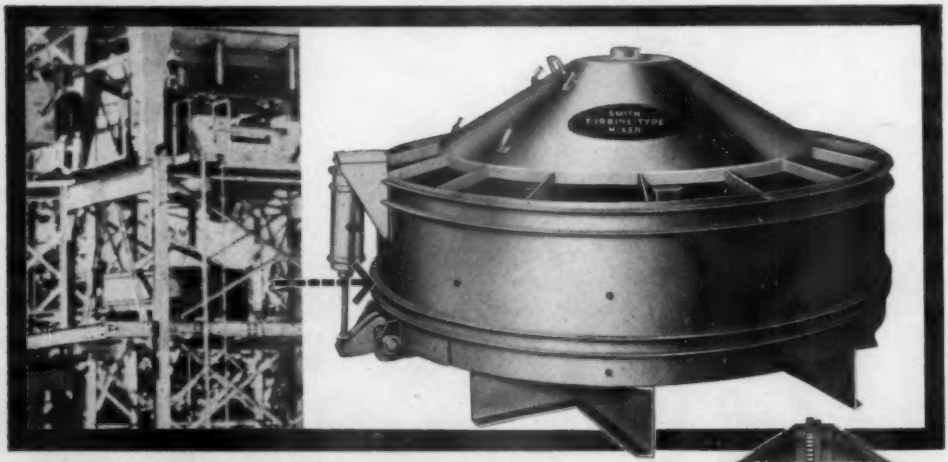

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LACLEDE 7-WIRE STRAND USED EXTENSIVELY IN PRESTRESSED CONCRETE SLABS FOR NEW YORK PIER

New Pier 40, a massive concrete structure 800 feet square, is taking form in New York harbor. When completed in 1960, it will replace five older finger piers and will be one of the most efficient ship handling facilities in the country.

Substructure of the pier consists of a prestressed concrete slab deck resting on poured-in-place concrete beams and caps, supported on steel H-piles.

Six thousand prestressed slabs were used to form the deck. They measure 1 ft. thick, about 5 ft. wide and 20-25 ft. long. A large portion of this structural material was pretensioned with Laclede 7-wire prestress strand.

General contractor for all substructure work on the pier was Stock Construction Corporation, under contract with the Department of Marine and Aviation, City of New York, whose engineers are Roberts and Schaefer Company.

Pretensioned slabs for new Pier 40 were subcontracted to Corbetta-Precrete. They were cast in off-site prestressing beds and delivered to the harbor jobsite ready for immediate installation.

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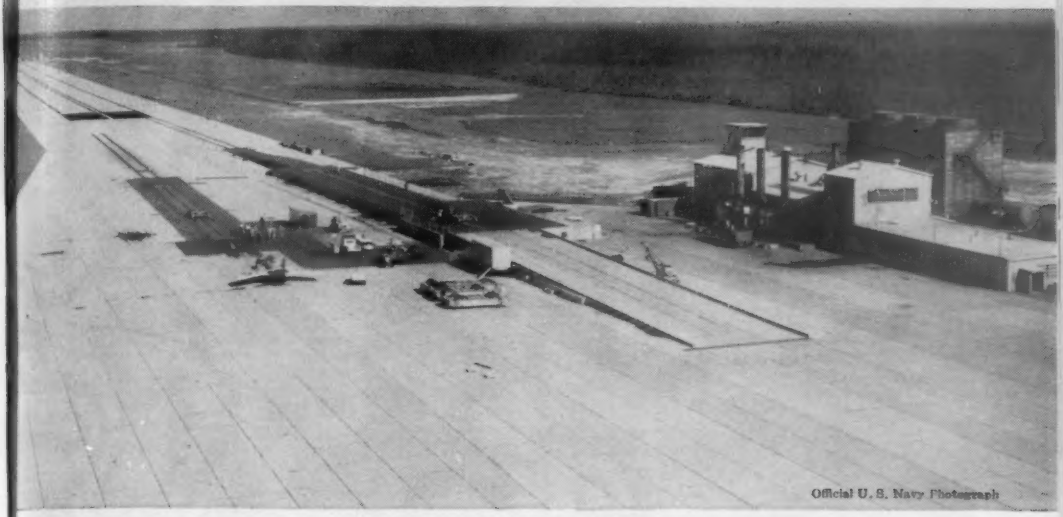

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NAVAL AIR TEST FACILITY—Lakehurst Naval Air Station, N.J. Construction Agency: Bureau of Yards and Docks of the Navy Department. Constructed under the direction of the District Public Works Officer, 4th Naval District, Philadelphia, Pa. • Consulting Engineer on Pavements: Louis Berger & Associates, Orange, New Jersey • Contractor: Roscoe Engineering Corporation & Associates, Washington, D. C. • Pavement Sub-Contractor: S. J. Groves & Sons Company, Woodbridge, N. J.

900 psi flexural strength— POZZOLITH helps meet this requirement at The Naval Air Test Facility

The Runway Arrested Landing Site of the Navy's recently completed $23,000,000 test facility required concrete of unusually high quality.

To withstand anticipated stress of jet planes simulating aircraft carrier landings, reinforced concrete having a flexural strength of 900 psi was specified.

Preliminary mix designs—with cement factors ranging up to 8½ sacks per cubic yard and with the water-cement ratio of 4.0 gallons per sack—did not provide a workable mix with the required high flexural strength.

The following mix was then employed:

Cement, Type 1—729 lbs.—(7¾ sks.)
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This mix was workable and developed a flexural strength of 951 psi in 7 days and 1114 psi in 28 days. Compressive strength averaged 8537 in 28 days.

Because of these high strengths the water-cement ratio was later increased to 3.75 and the cement factor reduced to 7½ bags. Flexural strength of the revised POZZOLITH mix averaged 981 psi in 28 days.

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This on-the-job photograph, taken at Eastern Pre-stressed Concrete Corp., Line Lexington, Pa., shows the arrangement of Bethlehem Stress-Relieved Strand in box beams. The beams were 68 ft long, 48 in. wide, and 33 in. deep. The initial stress induced was 14,500 lb per strand, using a 47-strand pattern with $\frac{3}{8}$-in. strand.

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Bethlehem Stress-Relieved strand exhibits excellent uniformity of mechanical properties, fulfilling every requirement of ASTM Specification A416-57T. For full details, get in touch with the nearest Bethlehem sales office.

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● Prestressed concrete has attained new heights in Seattle's 21-story Norton Building, first U.S. structure over six floors to utilize P/S. The unique design uses 238 P/S beams in combination with a steel perimeter frame to support column-free floors in the 17-story tower. Each 15-ton beam spans 70′, is only 37″ deep.

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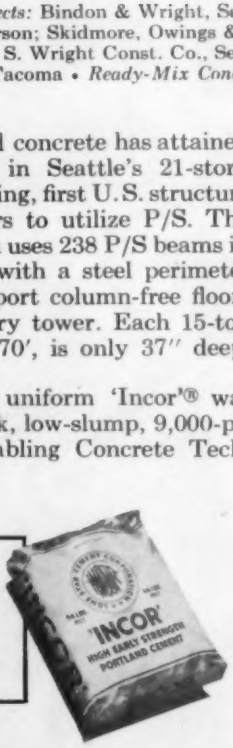

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DR. A. ALLAN BATES
Vice President for Research and Development PCA Association

GUEST EDITORIAL

■ Conceived a generation ago in the minds of a few inspired scientists of western Europe, the technology of prestressed concrete is emerging as a mass production industry, especially in countries like the United States and Russia, where mass productivity is a purposeful national necessity.

Prestressed concrete offers potentially the most efficient use of major structural materials which science has evolved. Happily, functional efficiency tends to carry its own visual attractions. Prestressed concrete, when well designed and carefully produced, therefore naturally embodies the two principal qualities, efficient use of materials, and aesthetic appeal, which have characterized all great architecture since the building of the Hellenic temples of 2500 years ago.

The full realization of these natural potentials will result, however, only from the continuous and cordial cooperation of all interested parties. The research and development laboratories of the materials and construction industries must aggressively improve the properties and extend the available knowledge of structural materials; design engineers must make bold and informed use of this advancing knowledge of materials capabilities; architects must conceive structural concrete in new terms of lightness and grace instead of massiveness and stolidity. And, finally, producers of this new, light, and graceful concrete — the precasting and prestressing industries — must be adventuresome in pioneering closely controlled low cost production methods.

All these groups are succeeding in their tasks in more than an ordinary manner. But the challenge is the extraordinary one of launching a new age in the history of architecture and construction.

The Research and Development Laboratories of the Portland Cement Association stand ready, able and eager to play their part in this cooperative evolutionary wave of architectural and structural future. To the members of the Prestressed Concrete Institute and others we send our best wishes for continued success and cordial assurances of our determination to help in every possible way.

EDITORIAL COMMENTS

It affords me personal pleasure to present our guest editorial by Dr. A. Allan Bates, Vice President for Research and Development, Portland Cement Association. For he spotlights two themes, not fully recognized by prestressors — first, the requirement that modern concrete meet esthetic demands as well as the older utilitarian demand; and, second, the necessity for close cooperation of many groups in the development of the great possibilities for prestressed concrete. In this connection, we hasten to proclaim at once the splendid cooperation and assistance given PCI by PCA — no one has manifested more zeal. We also acknowledge most happily, close cooperation being obtained with other organizations, among them the American Concrete Institute and the American Society of Civil Engineers — the latter scheduling a joint meeting with PCI for next spring. PCI is closing ranks with progressive groups everywhere and we are definitely on the march together. As to the requirement expounded by Dr. Bates that prestressed concrete meet esthetic demands. I cannot emphasize any too strongly the wisdom, the potentials for progress in that advice.

° ° °

Henning Collborg, who came from Sweden to address our recent Convention at Miami Beach, speaks to us here on prestressed concrete buildings of from one to fourteen stories. He is one of the earliest pioneers of that new material in Europe. His company holds an Active Membership in PCI which makes us doubly proud of his accomplishments, his long distance interest and participation.

° ° °

Try fitting a deflected strand pattern to a parabolic loading diagram, using mathematics only as a tool. Distress may sometimes follow. You will find here a graphic design method by William Wendland which determines the critical design point with simplicity and clarity. Graphs can be similarly constructed for any standard deflected strand product.

° ° °

A unique feature in the article on the Panama City Bridge is that it marks the first time the Florida State Road Department has employed hollow precast prestressed concrete cylinders for the substructure of a bridge. Moreover, prestress won out as an alternate design.

Read once again the second paragraph in Dr. Bates' editorial before indulging your pleasure in the space frames at the Gatwick Airport. There was no collusion between him and the airport's architect for attaining efficient use of materials and esthetic appeal. High level thinking and talents usually converge on pinnacles of art and science though separated literally by oceans as in this case. And new potentials bringing new markets to prestressing will appear with every step upward by the industry where their cold assembly line earth products are raised to architectural inspiring realms.

° ° °

We present with special pride three original papers from three different universities which should serve together as a ready-reference library of important data previously unavailable:

Dr. Ozell, University of Florida, teaches us "experimentally and analytically" the effects of shrinkage and creep in composite construction. The great Bill Dean's hand is visible in support of this project since it was sponsored by the Florida State Road Department.

° ° °

Dr. H. Brettle, University of New South Wales, presents a new deflection theory developed from good correlation between the theoretical and observed deflection curves of all beams tested. An extension of his paper "A Theory to Predict the Neutral Surface Position in Rectangular Prestressed Concrete Beams as Cracking Develops" was published in the Proceedings of the Second National Prestressed Concrete Conference held at Wellington, New Zealand, in 1958.

° ° °

Everyone should commit some time to study of the Durability Tests by Stanley Lee Rieb, Kansas State University. Manufacturing of prestressed concrete has brought about the integration of processing and combining of concrete and steel into a single product of much greater strength within less space than ever attained before in masonry. That is why the Institute assumed its personal responsibility to pursue these tests. This article serves effectively towards evaluation of the new product. Excellent and valuable conclusions resulted.

MARTIN P. KORN.

prestressed concrete building production in Sweden

by

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Aktiebolaget Strangbetong
Stockholm, Sweden

SYNOPSIS

The development of factory-produced prestressed concrete in Sweden is outlined, and five typical buildings are described: (1) a streetcar garage, (2) a single story industrial building, (3) a three-story department store, (4) a five-story industrial building, and (5) a fourteen-story office building. Details of connections, between precast members are emphasized, and adaptation for factory production is discussed.

DEVELOPMENT OF PRODUCTION

I would first like to review briefly how factory-produced prestressed concrete made its way into Swedish construction. During our first period of production in the early 1940's we had as our goal the introduction of the new material as roof beams for industrial buildings, for which the most direct technical and economical benefits were expected. Of course, there was a great deal of traditional thinking to overcome. Several years passed before we came to a fairly smooth-running production and before a good production standard had crystallized. The introduction of the prestressing techniques into the building market coincided with and satisfied the demands for larger spans and flexibility in planning associated with the industrial renewal in Sweden during and after the last world war.

The next phase of development came when we began to prefabricate for erection columns and primary beams also. It was soon found that there was an additional economic advantage in avoiding all regular site-concreting above ground level. Thus, our competitive power was increased through this growing development toward a wholly prefabricated structure.

Another stage of great importance occurred when we began to erect the members produced at our factory. Apart from other advantages, this was a step of progress because we became able to more systematically learn the faults and merits of many details and to improve them.

Having come to the stage of delivering whole frameworks for one-story buildings, it was tempting to go further with multi-story buildings. The first trials were again made with industrial buildings. Today, however, we have also started with structural frames for other multi-story buildings, such as office buildings and warehouses. However you must bear in mind that when we speak of multi-story buildings in Scandinavia, it is not the very high buildings of the United States, but buildings in the range of three to twenty-five stories.

Before leaving the history I must add that our developments of bridge construction have progressed more slowly. It is only in the recent years that large bridges have been taken into consideration.

FIVE TYPICAL BUILDINGS

Streetcar Garage

The first building I have selected to discuss is a garage for streetcars, (Fig. 1). It is a simple building with limited demands for daylight and spans. It is a typical example of warehouses, garages, and less complicated workshops. The building area is 143,000 sq. ft. (Fig. 2). The spans are in one direction 25 ft. and in the other 45 ft. The structural frame is wholly prestressed; the roof slabs as well as the wall panels are made of cellular concrete, conventionally reinforced. There are three transverse expansion joints.

I would like to now discuss the design of the details at various connections, shown at A, B, C, D, E, and F in Fig. 2. At the gable columns, see Fig. 3, the primary beams are placed on a steel plate to which the beams are connected by a vertical bolt. The hole for the bolt in the beam is of rectangular section and is spacious to accommodate deviations on beam length. The hole is, of course, filled with cement mortar after erection which is also the case for all recesses in the following connections.

At the internal columns, (See Fig. 4) the primary beams are guided laterally by specially designed "beam-guides" of steel plate anchored to the top of the columns. In order to improve stiffness, the steel plate is bent at the edges. When the beams are in place, the two sides are pulled together by a bolt. The beams are connected to each other by one or two left- and right-handed threaded screws which are dropped into specially designed hardware cast into the beam ends. Since some purlins are also coming in just over the column, the primary beams are cut off sloping to ensure a better support.

If we now consider the purlins, we find that they are placed fundamentally in the same way (Fig. 5). The columns of the side walls have similar steel plate "beam-guides" in the top. The beam is anchored here by a horizontal bolt going through the beam and the sides of the guide. Lowest in the picture we can see the design at an expansion joint. The support and anchoring of secondary beams or purlins, (Fig. 6), on the primary beams, are principally the same as for primary beams at columns.

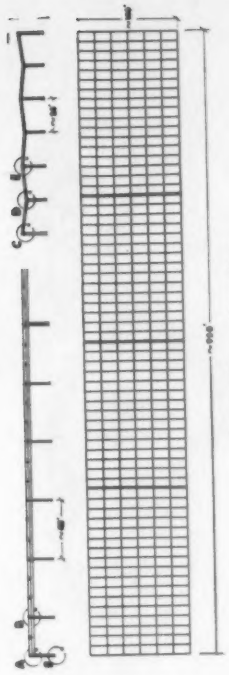

Fig. 2 Plan and sections

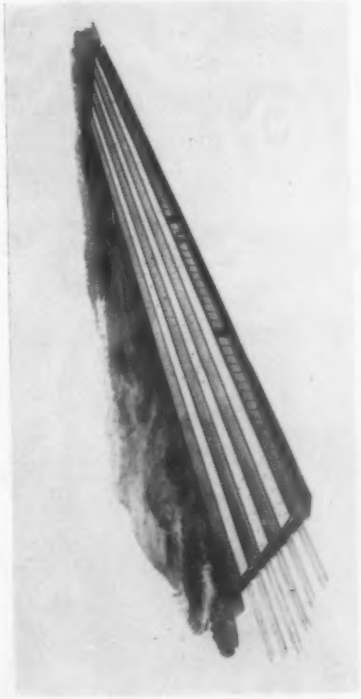

Fig. 1 Perspective of streetcar garage

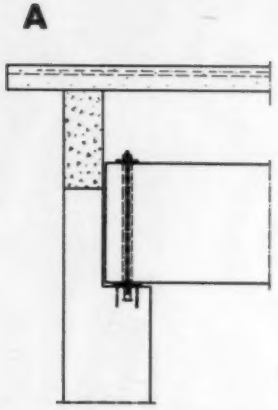

Fig 3 - Connection of primary beams to gable columns.

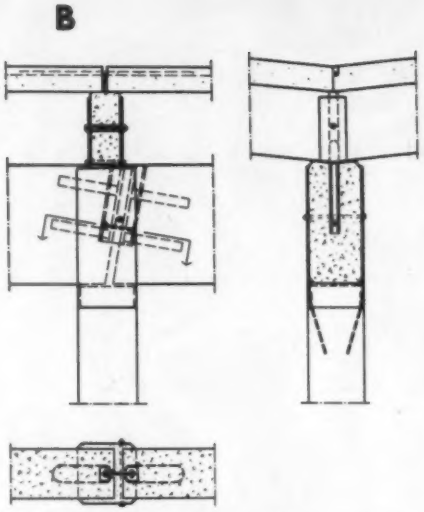

Fig 4 - Connection of the primary beams at the internal columns.

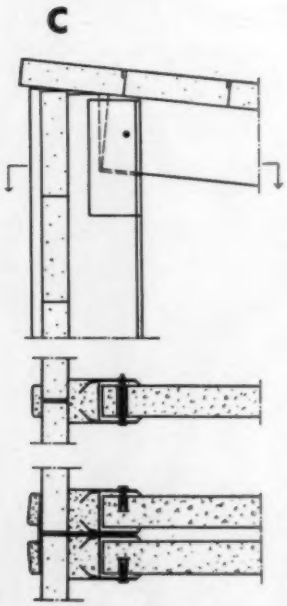

Fig. 5 - Placing of the purlins on the columns of the side walls.

We consider these U-shapes steel guides to be a pretty good solution. The beam connection is hidden, a satisfactory stability is obtained until the roofing is finished, no forming is required for the filling of cement mortar, and the entire joint appears clean and good looking. The guides cost a little extra, but they are facilitating the erection work and have proved themselves worth their price.

The cellular concrete roof slabs have a 20-in. width modulus. They are anchored to the purlins by means of thin steel rods protruding from the top purlin surface. The rods are bent over short bars placed in the grooves between the slabs. The grooves are filled with mortar before the asphalt roofing paper is laid.

Columns are commonly fixed at the foundation (Fig. 7). The concreting is done by the building contractor. At the base of the columns there is a type of adjusting screw to be adjusted after the bottom of the foundation pit is leveled. A pressure distributing plate is inserted in the pit meeting the screw.

The wall panels are clamped to the columns by a covering mullion of prestressed concrete. These mullions are held to the columns by bolts screwed from the inside, (see Fig. 7, section a-a). The blockout for the bolt is filled with grout. This wall construction is perhaps the cheapest form when heat insulation is demanded. If a hard wall surface is desired, some of the lower panels are replaced by insulated concrete panels.

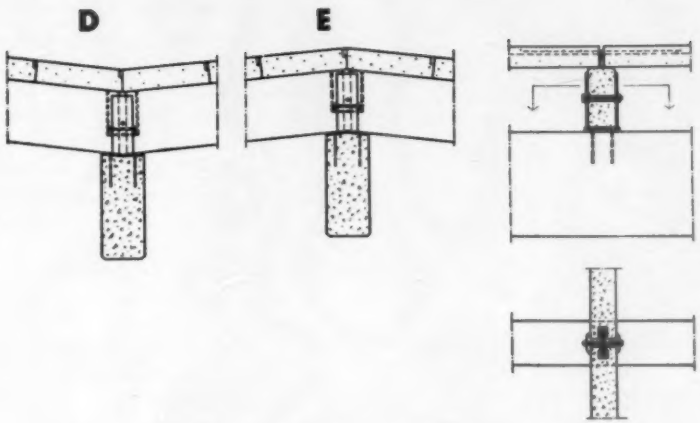

Fig. 6 Support and anchoring of the purlins on the primary beams

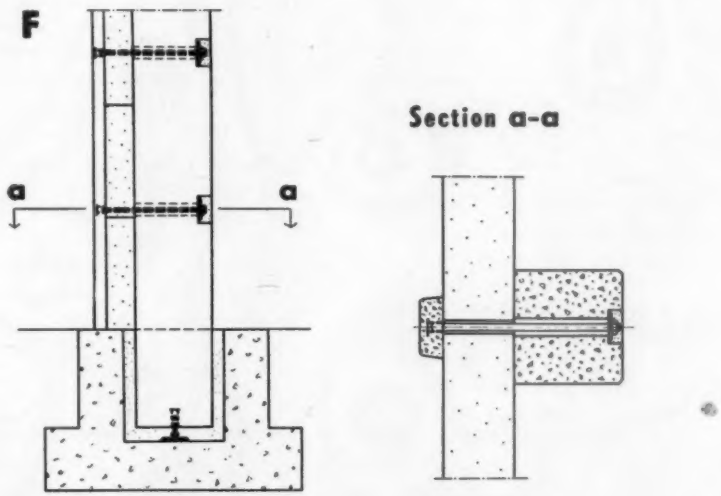

Fig. 7 Fixing of columns to foundation. Section a-a: Connection of wall panels to columns

One-Story Industrial Building

The next building to be described belongs to a mechanical industry where greater column spacings and better daylight conditions are required, (Fig. 8). This building is fairly typical of today's standard for larger factory buildings in Sweden. The facades are here built up of red facade-brick with the inner side of yellow brick without plaster.

The plan measures are 240 by 390 ft., and the spans are 40 and 80 ft. (Fig. 9). Columns are fixed to the foundation and have internal downspouts for drainage water. The beams are 18 in. wide and 43 in. high. They are I-shaped with solid end blocks. By means of a concrete topping they are made continuous for the live load. In this case there are two side-by-side to resist the heavy load from the roof and traveling cranes without becoming too deep. The space between beams is utilized for pipes and cables of various kinds. The roof is shaped as saw tooth. This type is commonly used in Sweden to have a good distribution of natural light. The roofing is again slabs of cellular light concrete. The secondary beams are designed as rigid frames. The slightly inclined part is bed-manufactured, while the almost vertical leg is conventionally reinforced.

We shall look a little closer at a few connections, (Fig. 10). To obtain very low column heads these are strengthened by two short steel joist. As said before, continuity is obtained by a concrete topping, in this case 4 in. thick. To avoid further complications in the joint above the column, the secondary frames are placed offset, (Fig. 11). The slightly inclined leg is made with a tap end fitting a hole in the beam web. The tap end has a steel reinforcement to which a supporting plate is connected. Moreover, bars are protruding to give a tension connection to the primary beams as the top concrete is placed down between the beams for a short length around the joint. The other leg is connected to the top flange by welding, or by bolts which tie together a steel plate in the leg and one cast into the top of the beam.

The two parts making the frame are joined to resist bending moments, (Fig. 12). Two pairs of high tensile steel rods are post-tensioned to 10 tons each. Before that, the space between is filled with dry cement mortar. Smaller roof frames are built together at the factory; larger ones on the building site before erection.

It may be of interest to see how this same roof frame is attached when the primary beam is a single one, (Fig. 13). The slightly inclined leg is then supported on a seat on the lower beam flange. The almost vertical leg is fastened after that the other part is placed snug to the beam web. Tolerances thus are taken up in the supports of the vertical leg. In order to have the beam insulated and to get a suitable shape of the frame valley to function as a gutter, a T-steel is placed horizontally between the frame and beam to carry the roof slabs. The T-steel also functions as a tension tie between the two concrete members.

An alternative H-shaped beam profile is often used for saw-tooth roofs, (Fig. 14). This profile is very practical. The roof channel is used as a platform when window frames and roof slabs are erected. Roof slabs are, in this case, standing and supported directly against a window frame made of concrete. The H-beam has such a rigidity that no roof frame is needed. The roof channel is insulated with cork. The lower channel can be used as ducts for warm air heating, for pipes, conduits, and so on. It is covered with perforated aluminum sheets.

I shall complete discussion of this building with two photos from the time of construction, (Figs. 15 and 16). The last photo shows the stage when the prestressed concrete supplier is handing over the completed framework to the general contractor after a joint inspection. The time required for erection was six weeks for a 3-man crew plus a foreman.

Three-Story Department Store

In a new suburb of Stockholm, there is at present under construction a shopping center consisting of four buildings all designed as structural frames of prefabricated prestressed members. We shall consider one of them, (Fig. 17).

The base of the building is 150 by 200 ft., (Fig. 18). The spacing of the columns is 30 by 30 ft. The irregular shape of the building is, of course, not the very best

Fig. 8 View of one story industrial building

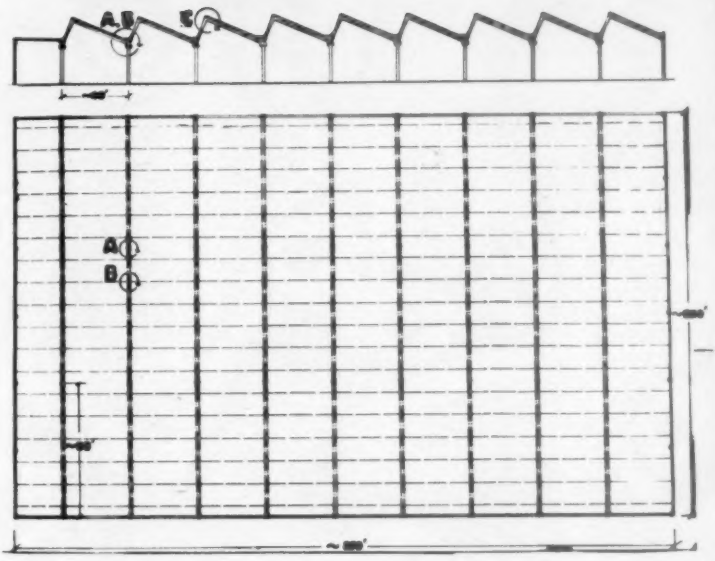

Fig. 9 Plan and section

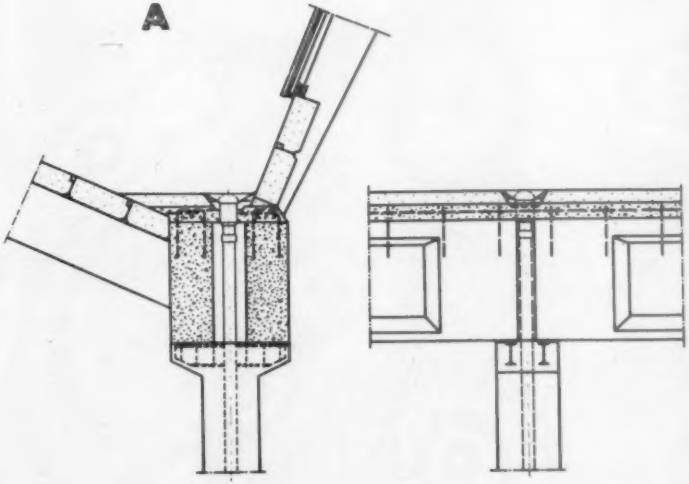

Fig. 10 Support of roof beams on internal columns. Continuity obtained by concrete topping

PRINT IN BINDING

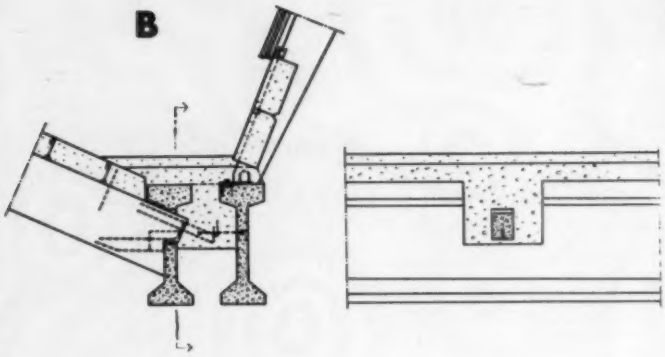

Fig. 11 Connection of the secondary frames to the roof beams

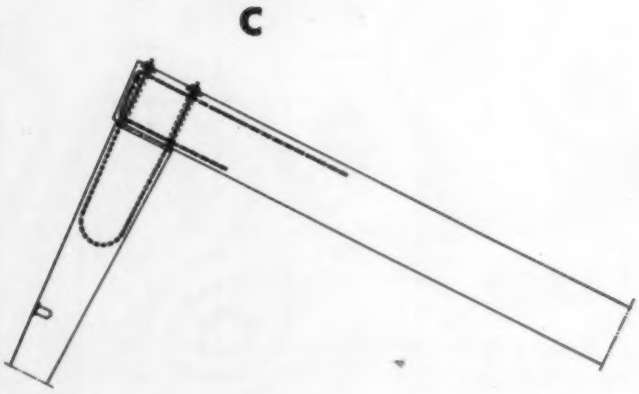

Fig. 12 Joining of the two legs of the frame

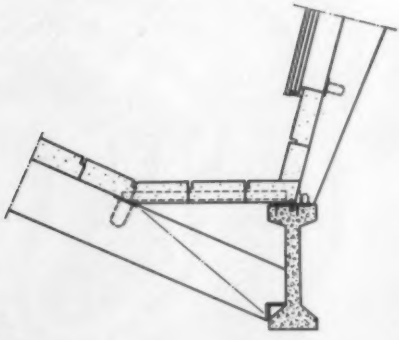

Fig. 13 Attach of the roof frame to a single primary beam

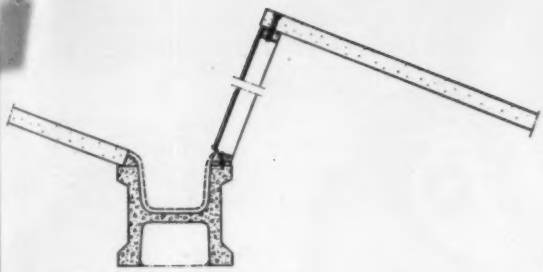

Fig. 14 H-shaped beam profile for saw-tooth roof

Fig. 15 View from the time of construction

Fig. 16 The framework completed by the supplier

from the manufacturer's point of view. In the middle of the building are the escalators and at the sides, elevators and staircases. No other moveable partition walls are shown. The columns, except some in the facade, are U-shaped with a groove for utility devices. In some of them there are drainage pipes, in others fresh water pipes, electrical cables, etc. Utility lines within the stories are placed between the floor slab and a suspended ceiling. The outer line marks a cantilever canopy going around the building.

The sections in Fig. 19 show how the structural frame is made up of columns, rectangular beams and double T-slabs. The slabs as well as the beams have a situ-cast concrete topping functioning together with the prefabricated members as composite construction. Yet the slab is so calculated that, for changes of the building in the future, it will be possible to take away single slabs without losing too much load-carrying capacity. As shown in the section in Fig. 19, a walkway comes in at the second floor. The traffic goes under. The ground floor is used mostly as storage for the shops. The roof is also made as an ordinary floor slab, since it is foreseen that additional stories will be added in the future. The roof, therefore, is a light wood panel construction with sheet metal roofing.

If we take away walls and slabs we will see the column and beam pattern better, (Fig. 20). A design was chosen with primary beams in one direction only, thus making it possible to erect all members from the ground level. The mobile crane goes in one bay at a time, and places the slabs for all floors simultaneously. In this case the foundation did not permit the columns to take the wind moments during erection. Therefore, temporary cross bracings were required during the erection.

The frame connection is illustrated in Fig. 21. At the column bracket, the groove changes to a hole making it possible for the bracket reinforcement to extend through.

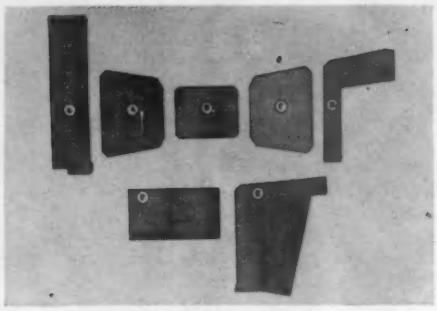

Fig. 17 General plan

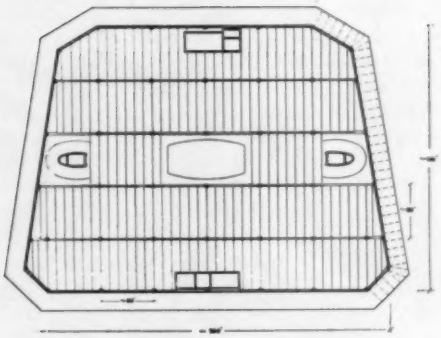

Fig. 18 Plan of the building "C"

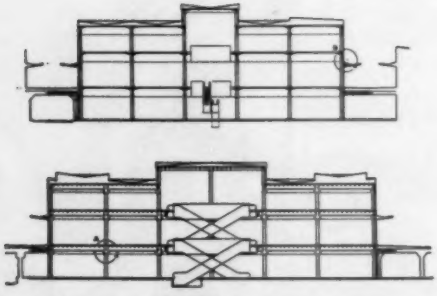

Fig. 19 Sections

The floor beams are connected to the columns by vertical bolts. T-slabs have the slab coming down to the bottom of the stems at the ends. This has two effects; one is that the concrete stress in the stems is lowered in order to resist the combined stress from pretensioning and from the negative moment, the other is that all forming between the stems is avoided when the topping is cast. Furthermore, this design can resist heavier shear forces. This type of T-slab is only used where continuity will be established. In simply supported spans, we make them with a constant cross-section. The columns go through the connection, and the continuity reinforcement of the floor beams must, therefore, be placed inside the columns, which is easily possible due to the shape of the T-slabs.

The support at a wall line is shown in Fig. 22. The steel framing carrying the cantilevered canopy is anchored in the floor slab topping, and is bolted below to a special spandrel beam. This structure can, without complications, be adopted for wall panels of concrete as well as other wall types, such as brick, cellular concrete, etc.

At openings in the floor slab, edge beams are used, (Fig. 23). They are usually supported by blockouts in the primary beams. In order to obtain a finer modulus than the T-slab width, half-slabs can also be delivered. The standard width is 5 ft. Three stem-heights are manufactured, 12, 14 and 16 in.

Some photos with glimpses from various stages of the construction work are shown in Figs. 24a, b, c.

Five-Story Industrial Building

Another way of solving the connection between beams and columns, for which good ability to resist moment has been obtained is shown in Figs. 25, 26. The columns are cast in one piece, and are so reinforced and prestressed that they are capable of taking the wind loads during erection without temporary bracings. The floor beams are first freely supported, placed on the fork-shaped insert device cast into the columns. The stud bolt below is placed from the outside through the columns and is then bolted or welded to a pair of ears extending from the beam end. When the floor slabs are concreted over the prestressed planks, the space between beam and column is also concreted. A prestressing cable or

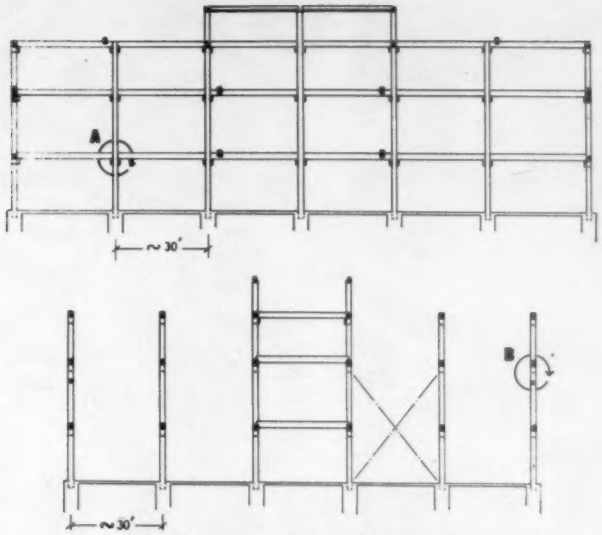

Fig. 20 Sections, columns and beam patterns

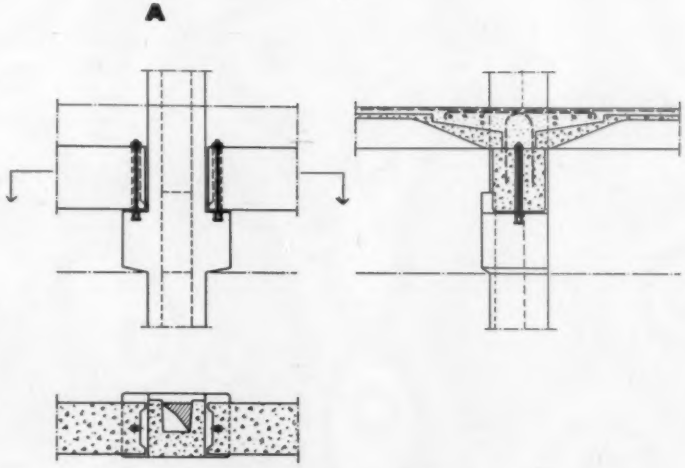

Fig. 21 A frame connection

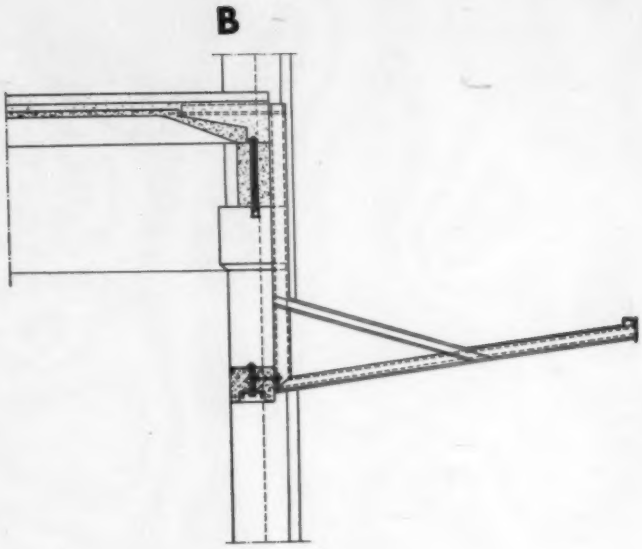

Fig. 22 The support at a wall line. Anchoring of the cantilevered canopy

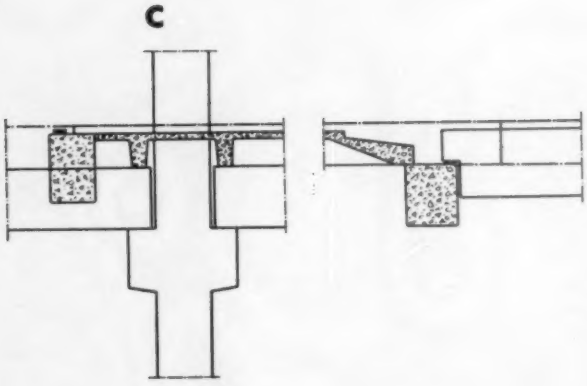

Fig. 23 Edge beams at floor opening

bar is placed above each deck beam. It is tensioned after the top concrete has hardened, and the stud bolt below is tightened. The structure is quite stable, the staircases and elevator shafts are not designed to stabilize the building. All members are placed from the ground level. After putting up the columns, the erection goes on, bay after bay.

During construction the building appeared as seen in Fig. 27a, and completed as shown in Fig. 27b. Wall panels are built up of wood and rockwool with gypsum plates inside and aluminum cover outside.

Fourteen-Story Office Building

Though this reader audience is accustomed to tall buildings, I would like to show the tallest prefabricated building hitherto erected in Scandinavia and perhaps in Europe, (Fig. 28). It is a 14-story office building. The building is divided in four parts, erected as separate stages, and stacked on top of each other. The stability was attained by assistance of the two staircases and elevator shafts at the gable walls. The columns are spliced by means of a very well anchored steel collar at each end tied together with high tensile bolts, (Fig. 29).

For erection, the columns are balanced on a small steel plate in the center. In this way it is easy to plumb the columns exactly. Deviations in column length are taken up by varying the thickness of the little steel plate. After casting the floor-topping concrete, the space between column plates is injected with cement grout in the same manner as prestressing cables. One pipe is for filling, the other for airing. The beams carrying the channel slabs are exposed in the final building. The ceiling is placed just below the slabs. The column brackets are therefore of structural steel, over-dimensioned to increased fire resistance. The beams are notched for the supporting steel flanges and later plastered. Beams are further connected to each other by bolts as described previously.

After erection of each part, the concrete topping of the floor slabs had to be placed before the next part could be erected. The column cross section was changed for the top two parts from 16 by 16 in. to 12 by 12 in. The upper parts consist only of three and two stories due to the fact that the available crane limited the weights.

The finished building is shown in Fig. 30. The facade is made up of panels in white concrete. They were delivered by a Danish firm, while the company I represent manufactured and erected the structural frames.

We may ask ourselves what obstacles there are to be met in going still further in height with factory-manufactured buildings of prestressed concrete. We think it is possible to build taller. Lateral stability will have to be provided by means of elevator shafts and stair-cases cast in situ and standing as poles in haystacks. Probably these shafts and staircases should be cast with slip forms, as a silo. The building can then be constructed around these cores. There is a limitation to be found in the column dimensions. They will be too wide and will take too much of the floor surface. A combination with steel columns may be possible, as it was done in Seattle's "Northon" building.

When good, economical solutions are found for really tall buildings, an enormous market for our line of industry will be opened up. I would like to vary Mr. Luckman's theme at the last meeting in Chicago and say "the future is in the sky."

Fig. 24a Department store during construction

Fig. 24b Floor Slabs before concrete topping is cast

Fig. 24c Floor slabs from below

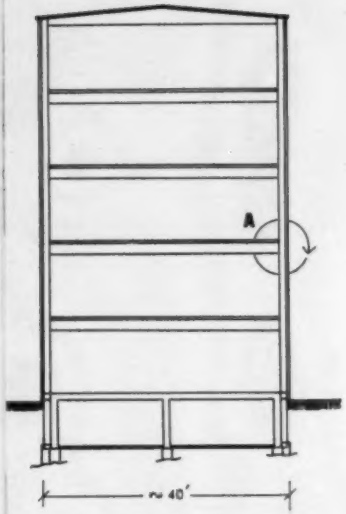

Fig. 25

Fig. 25 Section of five-story industrial building

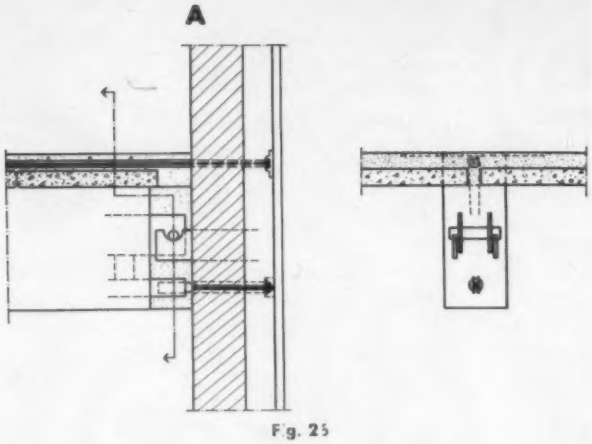

Fig. 25

Fig. 26 Support and connection of the floor beams to columns

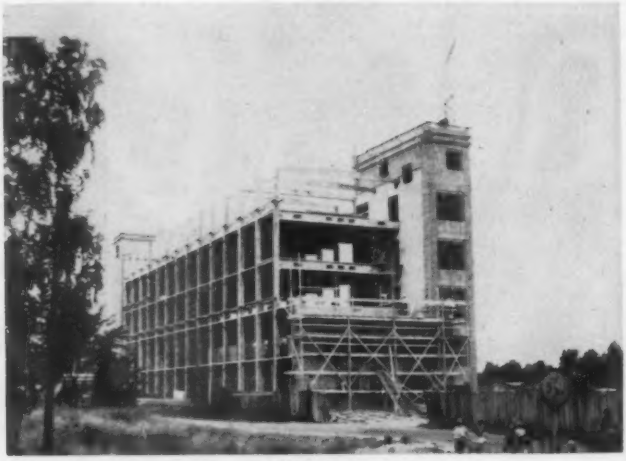

Fig. 27a

Fig. 27a From the construction time

Fig. 27b

Fig. 27b The building completed

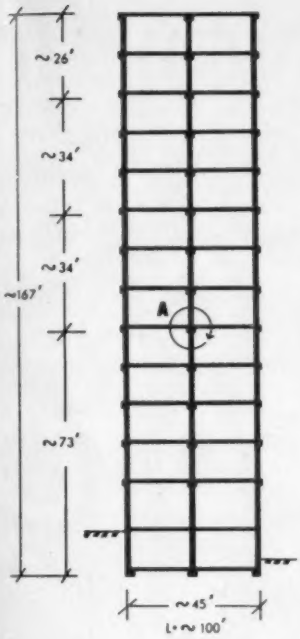

Fig. 28 Section of 14-story office building

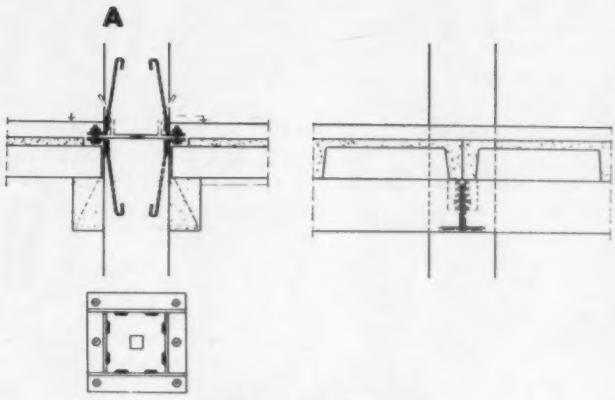

Fig. 29 Splicing of columns

ADAPTATION FOR FACTORY PRODUCTION

All prestressed members I have discussed here were pretensioned by the long-line method. Other companies in Sweden are using both pre-tensioning and post-tensioning. In our company we have utilized the former from the very beginning. When expanding our production facilities we have carefully compared the economical characteristics of this principle with others, but we have not found cause to change.

Our prestressing beds are from 100 to 300 ft. long. The main production is done on 250 to 300 ft. beds. Perhaps these beds appear short to you, but we have experienced that with regard to ranges for overhead cranes, walking distances, etc., the most favorable length is roughly 300 ft. as an average for most products. Certain members, such as T-slabs, are considered to be produced a little better on beds of 400 ft. length.

We cure the concrete in air, by steam, or in warm water. For air-curing, the beds

Fig. 30 Finished building

are wide to admit several castings side-by-side. All production is carried on indoors, and the costly bed area can, in our opinion, be utilized best if it is not divided up.

All forms for standard profiles are made of steel. Non-standard products are cast in wood forms. Rectangular members are normally cast in gang forms, using thinner steel forms for the interior walls. I-sections and other profiled members are made in a single row. Cross sections and beam reinforcement are standardized, but the lengths are custom made. Forms are so made that they, to some extent, can be used for several profiles.

The worst problem in adaptation for fabrication is what we call the details, of which I have given several examples. We are utilizing several production principles. The inserts and the blockouts may be attached to the form, or sometimes to the reinforcement in combination with special stud bars to the form. Other details are positioned during concreting, or they are applied wholly or partially after the member has left the casting bed.

Our design department utilizes standard solutions with typical drawings for all frequently encountered structures. However, new cases often appear and development, revision, and improvement are continuous features of our design work.

In general, I wish to emphasize at this point that it is extremely important for the inherent economy to find good solutions, not only statically, but also from a manufacturing point of view. This does not always mean that a solution which is cheapest in the production is finally the most economical. For example, an expensive insert can have a great influence on erection time or on the design of adjoining structural parts. A good example of this is the columns of the department store shown earlier. In that building, a lot of problems were intentionally concentrated in the columns to get the beams and slabs more "clean."

It happens sometimes that the consulting engineers must permit some changes in their designs in favor of production economy. Frankly speaking, we nevertheless have learned that in most cases the views of the architect, engineer, and manufacturer can well be merged together if all parties concerned are willing to cooperate fully. The question is merely, here as in many other endeavors, that the architectural form must pay a little regard to building methods.

GRAPHIC METHODS FOR THE DESIGN OF PRESTRESSED CONCRETE DOUBLE TEE PANELS

by
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PART I

INTRODUCTION

THE design of pretensioned prestressed concrete members having deflected or harped strands present some problems to the designer in that fibre stresses caused by dead and supermposed loads, when plotted, are parabolic in shape whereas fibre stresses caused by prestress show as straight lines. This is more clearly shown in Figure 1.

Let us consider for the time being only the bottom stress. Adequate design would be present when values for prestress, at any point on "L", are equal to or greater than values for bottom fibre stress due to dead and superimposed loads. (Tension allowed in bottom fibre will be discussed in a later section.) It would be simple for the designer to assign values for prestress which he knew would exceed the values of Mc ÷ I. However, this would generally be an uneconomical design and would likely produce certain excessive concrete stresses. It is, therefore, desirable to find some point "X" where the value for prestress is equal to Mc ÷ I, this being the point where the line is tangent to the curve, (Figure 1). There are too many unknown values for point "X" to be solved by means of

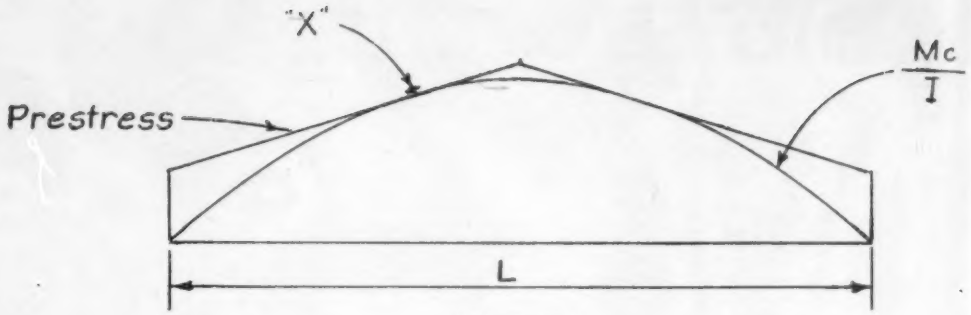

Figure 1.

simultaneous equations. Thus a means of graphic analysis becomes both necessary and practical. For the designer to plot values graphically for prestress and Mc ÷ I each time he is confronted with a design would be a tedious job. When standard sections are being used time after time, graphs can be readily developed which can be used to produce rapid and reasonably accurate results.

The accompanying set of curves was developed for bottom fiber stresses in a double tee panel of the cross section shown in Figure 2. Only simple spans have been considered. Top fibre stresses for this section seldom present much of a problem and therefore curves for top fibre stress have not been included. Designers who are not familiar with this section should use caution however.

The section properties shown in Table 1 have been computed many times by many engineers. Therefore, the calculations determining these properties are not included as a part of this discussion. It should be pointed out, however, that for convenience the values shown in subsequent calculations are for one stem only or one-half of the double tee. (See Appendix A)

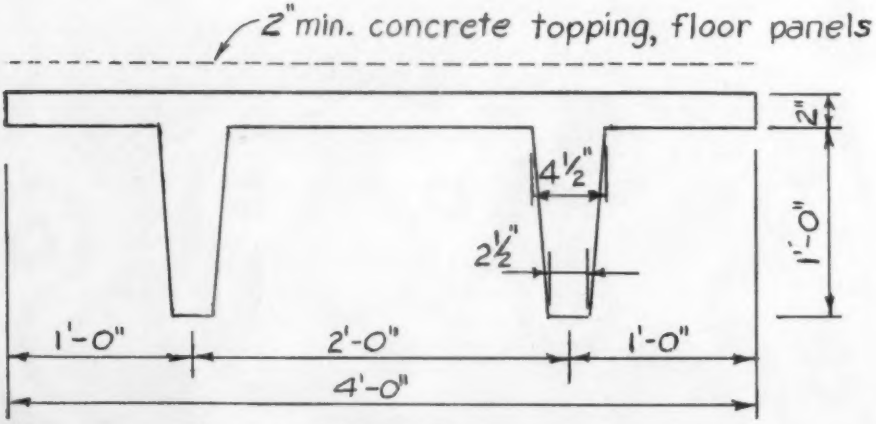

Figure 2.

Part II

BASIS AND DEVELOPMENT OF GRAPHS

SYMBOLS

Symbols used in this report are those in general use by designers in this area. They are noted below for convenience in following the remainder of the discussion.

f_b = bottom fibre stress (concrete)
f_t = top fibre stress (concrete)
f'_{ci} = compressive strength of concrete at time of transfer of prestress
f'_c = compressive strength of concrete at 28 days
P' = ultimate strength of prestressing steel
P_i = initial prestressing force before losses
P_o = prestressing force immediately after transfer
P = prestressing force after all losses
e = eccentricity of prestressing force at mid-span
e' = eccentricity of prestressing force at end of span
A = area of concrete section
S_t = section modulus about top fibre (without topping)
S_b = section modulus about bottom fibre (without topping)
S_{te} = section modulus about top fibre (composite section)
S_{be} = section modulus about bottom fibre (composite section)
w = uniform load per unit of length
DL = dead load
SL = superimposed load
TL = total load
M_{DL} = moment due to dead load
Ms_L = moment due to superimposed load
M_{TL} = moment due to total load

CRITERIA

Allowable steel and concrete stresses used are those shown in Section 207.1, of "Tentative Recommendations for Prestressed Concrete," ACI-ASCE Joint Committee 323. Compressive strength of concrete at 28 days has been taken as 5500 p. s. i.

GRAPH "A"

The lines on this graph represent the bottom fibre stress, at mid-span due to dead load of the Double Tee panel plus superimposed load, at various spans. The function of the lines is linear, being $f_b = \frac{M_{TL}}{S_b} = \frac{(DL + SL)L^2}{8S_b}$.

Two fibre stresses were calculated for each span shown and these points connected by a straight line. For the roof panels, the superimposed loads, used to calculate the two points, were 30 p. s. f. and 50 p. s. f. For the floor panels, the superimposed loads were 40 p. s. f. and 120 p. s. f.

Table 2 and Table 3 show the tabulation of the calculations for determining bottom fibre stress due to dead and superimposed loads.

GRAPH "B"

For uniform dead and superimposed loads, moment diagrams and corresponding flexural stress diagrams are parabolic in shape. Thus Graph "B" was plotted for a series of bottom fibre stresses (due to DL + SL) using 200 p. s. i. increments of fibre stress at mid-span. Table 4 (see Appendix A) shows the tabulation of the calculations used to develop the curves on this graph. Once plotted, a set of curves such as these can be used for any shape or section.

GRAPH "C"

This graph gives the bottom fibre stress due to the prestress caused by various numbers and sizes of strand at various eccentricities. The function of these lines in linear and so two points were determined, for each combination of strands, and these points then connected by a straight line. A third point served as a check in the accuracy of calculating and plotting the points, this point being where $\frac{P}{A} + \frac{P_e}{S_b} = 0$. All lines converge at this point. Table 5 shows the tabulation of the calculations for determining the above described points.

The dashed horizontal lines on Graph "C" give the required release strength at ends of the panels. These lines were determined from the formula $f_b = .60\ f_{ci}$, where $f_b = \frac{P_o}{A} + \frac{P_{oe}}{S_b}$ In determining P_o, it was assumed that one-third of all losses occur immediately upon transfer of prestress. For example, for one 7/16 inch strand $P_o = 18{,}900\text{-}1/3\ (18{,}900\text{-}15{,}120)$ or $P_o = 1.166\ P$. Table 6 (see Appendix A) gives the calculations for determining this function.

It should be noted that the zero ordinate for Graph "C" is located above the zero ordinate for Graph "B" in the amount of 445 p. s. i. The design f'_c for the Double Tee panels being 5500 p. s. i., the allowable tension in the bottom fibre under full dead and live loading is $f_b = 6\sqrt{f'_c}$ or 445 p. s. i.

GRAPH "D"

Upon transfer of prestress, the Double Tee panels camber upward and become simply supported at the ends. Thus the bottom fibre stress at center of span becomes, $f_b = \frac{P_o}{A} + \frac{P_o e}{S_b} - \frac{Ms_L}{S_b}$. As the span increases and the value of $\frac{P_o}{A} + \frac{P_o e}{S_b}$ remains constant, f_b diminishes. Graph "D" was developed using these conditions, the allowable f_b again considered to be $0.60\ f'_{ci}$. This function is also linear and so two points were determined and then connected by a straight line. The tabulation of the calculations is shown in Table 7. (See appendix A.)

PART III

USE OF GRAPHS

GENERAL

This series of graphs may be used to determine the required number and size of strand, their eccentricity at center and end of span, and the required concrete compressive strength at time of transfer of prestressing force, for a given span and a given superimposed load. (See typical example on graphs.)

Also, given a certain number and size of strand and their eccentricity at center and end of span, the allowable superimposed load may be determined along with the required compressive strength of the concrete at time of transfer.

Selections made from these graphs should be checked for required safety factor at ultimate strength.

MAKING A LOADING TABLE

A loading table can be derived by using the graphs also. This can be done by working "backwards" from a given f'_{ci} (minimum allowable concrete strength at release).

Starting in Graph "D" for a given span and a given f'_{ci} (for example 4000 p.s.i.), the maximum amount of prestress can be determined which will not produce a bottom fibre stress (at center of span) in excess of $0.6\ f'_{ci}$. This value is then transferred to Graph "C". Generally more than one strand pattern can be used; the magnitude of the eccentricity being the variable.

The next step is to choose the location of the strand at the end of the panel which will also produce a bottom fibre stress less than 0.6 f'_{ci}. In this case, no advantage can be taken of the reduction in bottom fibre stress due to dead load. Thus the allowable

$$f_b = 0.6\ f'_{ci} = \frac{P_o}{A} + \frac{P_o e'}{S_b}$$

The location of strand at the end of the panels is controlled by physical features of the casting bed. (Possible strand patterns are shown above the graphs.)

Values of prestress for end and center of panel, determined by the above described methods, are then transferred to their proper positions in Graph "B" and connected by a straight line. This straight line thus becomes a "prestress line" and represents the prestress at any point in the length of the panel. A "load stress curve" can be chosen which will be tangent to this "prestress line". It should be noted again that 445 p.s.i. tension has already been allowed in the plotting of Graphs "B" and "C". Thus the "prestress line" is actually at a value of 445 p.s.i. below the "load stress curve". The point where the line is tangent to the "curve" is the critical point in the design, being the point where final stress under design load reaches the maximum allowable tension.

The ordinate of the "load stress curve" at the center of the span (0.5L) determined by the above method, is then transferred to Graph "A". The point where the ordinate falls on the line representing bottom fibre stress for the span under consideration thus determines the maxium superimposed load that can be carried.

All values obtained by the above method of graphic analysis of elastic design must be checked against values obtained by ultimate strength design for use in tables; the lesser value controlling.

ACKNOWLEDGMENT

The author wishes to give credit to William H. Hebets for his ideas, encouragement and constructive comments during the preparation of this material.

APPENDIX "A"

TABLE I

SECTION PROPERTIES

TYPE	Area $in.^2$	Weight, lb/ft^2	I, $in.^4$	Y_b, in.	Y_t, in.	S_b, $in.^3$	S_t, $in.^3$
Roof Panel	180	46	2860	10.0	4.0	286	716
Floor Panel	270	71	4277	11.5	4.5	372	959

Note: Section properties for floor panels are based on a composite section with a two inch concrete topping having a 28-day compressive strength of 4000 p. s. i. A transformed section was used due to the difference in 28-day strengths of double tee panel and the two inch topping, following the "Tentative Recommendations" values for modulus of elasticity.

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This graph gives the bottom fibre stress due to the prestress caused by various numbers and sizes of strand at various eccentricities. The function of these lines in linear and so two points were determined, for each combination of strands, and these points then connected by a straight line. A third point served as a check in the accuracy of calculating and plotting the points, this point being where $\frac{P}{A} + \frac{P_e}{S_b} = 0$. All lines converge at this point.

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TABLE 2

BOTTOM FIBRE STRESS DUE TO DEAD AND SUPERIMPOSED LOADS (ROOF PANELS)

(1) L	(2) $\frac{L^2 \times 1.5}{143.2} =$	(3)	(4)	(5)
		Col. (2) x (SL + DL)		
		SL = 30 psf	SL = 40 psf	SL = 50 psf
24	6,033,519	923		1164
26	7,081,005	1076		1360
28	8,212,290	1235		1559
30	9,427,374	1434		1812
32	10,726,256	1630		2059
34	12,108,938	1841		2325
36	13,575,419	2063		2605
38	15,125,698	2299		2904
40	16,759,776	2547		3218
42	18,477,653	2809		3548
44	20,279,329	3082		3894
46	22,164,804	3369		4256
48	24,134,077	3668		4634
50	26,187,150	3980		5028
52	28,324,021	4305	4872	
54	30,544,692	4643	5254	
56	32,849,160	4993	5650	
58	35,237,429	5356	6061	
60	37,709,495	5732	6486	

Sample Calculation:

Assume Double Tee for 40 ft. span with 50 p. s. f. superimposed load.

$$f_b = \frac{M_{dl}}{S} = \frac{\frac{2\,(50 + 46)\,40^2 \times 12}{8}}{143.2} = 3218$$

APPENDIX "A"

TABLE 3

BOTTOM FIBRE STRESS DUE TO DEAD AND SUPERIMPOSED LOADS (FLOOR PANELS)

(1) L	(2) 1.5 L^2	(3) Col.(2)x(DL) / S_b DL = 71 psf	(4) Col.(2)x(SL) / S_{bc} SL = 40 psf	(5) Col. (3) + Col. (4)	(6) Col.(2)x(SL) / S_{bc} SL = 120psf	(7) Col. (3) + Col. (6)
18	486	482	210	692	629	1111
20	600	595	259	854	777	1372
22	726	720	313	1033	940	1660
24	864	857	373	1230	1119	1976
26	1014	1006	438	1444	1313	2319
28	1176	1166	508	1674	1523	2689
30	1350	1339	583	1922	1749	3088
32	1536	1523	663	2186	1989	3512
34	1734	1719	749	2468	2246	3965
36	1944	1928	839	2767	2518	4446
38	2168	2150	936	3086	2808	4958
40	2400	2380	1036	3416	3108	5488

SAMPLE CALCULATION:

Assume Double Tee for 30ft. span with 40 p. s. f. superimposed load.

$$f_b = \frac{M_{DL}}{S_b} + \frac{M_{sL}}{S_{bc}} = \frac{\frac{2(46+25)30^2}{8} \times 12}{143.2} + \frac{\frac{2(40)30^2}{8} \times 12}{185.3} = 1339 + 583 = 1922 \text{ psi}$$

TABLE 4 — STRESS CURVES

(1)	(2) .05L	(3) 0.10L	(4) 0.15L	(5) 0.20L	(6) 0.25L	(7) 0.30L	(8) 0.35L	(9) 0.40L	(10) 0.45L	(11) 0.50L
	Col. (1)x 0.475	Col. (1)x 0.09	Col. (1)x 0.1275	Col. (1)x 0.16	Col. (1)x 0.1875	Col. (1)x 0.21	Col. (1)x 0.2275	Col. (1)x 0.24	Col. (1)x 0.2475	Col. (1)x 0.25
4000	190	360	510	640	750	840	910	960	990	1000
4800	228	432	612	768	900	1008	1092	1152	1188	1200
5600	266	504	714	896	1050	1176	1274	1344	1386	1400
6400	304	576	816	1024	1200	1344	1456	1536	1584	1600
7200	342	648	918	1152	1350	1512	1638	1728	1782	1800
8000	380	720	1020	1280	1500	1680	1820	1920	1980	2000
8800	418	792	1122	1408	1650	1848	2020	2112	2178	2200
9600	456	864	1224	1536	1800	2016	2184	2304	2376	2400
10400	494	936	1326	1664	1950	2184	2366	2496	2574	2600
11200	532	1008	1428	1792	2100	2352	2548	2688	2772	2800
12000	570	1080	1530	1920	2250	2520	2730	2880	2970	3000
12800	608	1152	1632	2048	2400	2688	2912	3072	3168	3200
13600	646	1224	1734	2176	2550	2856	3094	3264	3366	3400
14400	684	1296	1836	2304	2700	3024	3276	3456	3564	3600
15200	722	1368	1938	2432	2850	3196	3458	3648	3762	3800
16000	760	1440	2040	2560	3000	3360	3640	3840	3960	4000
16800	798	1512	2142	2688	3150	3528	3822	4032	4158	4200
17600	836	1584	2244	2816	3300	3696	4004	4224	4356	4400
18400	874	1656	2346	2944	3450	3864	4186	4416	4554	4600
19200	912	1728	2448	3072	3600	4032	4368	4608	4752	4800
20000	950	1800	2550	3200	3750	4200	4550	4800	4950	5000
20800	988	1872	2652	3328	3900	4368	4732	4992	5148	5200
21600	1026	1944	2754	3456	4050	4536	4914	5184	5346	5400
22400	1064	2016	2856	3584	4200	4704	5096	5376	5544	5600
23200	1102	2088	2958	3712	4350	4872	5278	5568	5742	5800
24000	1140	2160	3060	3840	4500	5040	5460	5760	5940	6000
24800	1178	2232	3162	3968	4650	5208	5642	5952	6138	6200
25600	1216	2304	3264	4096	4800	5376	5824	6144	6336	6400

The values for Col. 11 are arbitrary. For ease of computation, Col. 1 is made to be 4 times Col. 11.

$$M_x = \frac{wx}{2}(L - x)$$

For $x = 0.5L$, $M_x = \frac{w(0.5L)}{2}(1 - 0.5L) = \frac{w \times 0.25L^2}{2} = C \times 0.25$

For $x = 0.2L$, $M_x = \frac{w(0.2L)}{2}(1 - 0.2L) = \frac{w \times 0.16L^2}{2} = C \times 0.16$

TABLE 5

No. and Size of Strand	P	P/A	$\frac{Pe}{S_b}$, e = 8″	P/A + Pe/S_b
1-7/16″	15,120	168	844	1012
2-7/16″	30,240	336	1689	2025
3-7/16″	45,360	504	2534	3038
4-7/16″	60,480	672	3379	4051
5-7/16″	75,600	840	4223	5063
6-7/16″	90,720	1008	5078	6076
1-3/8″	11,200	124	626	750
2-3/8″	22,400	249	1251	1500
3-3/8″	33,600	373	1877	2250
6-3/8″	67,200	747	3753	4500
1-3/8″ & 1-7/16″	26,360	293	1473	1766
2-3/8″ & 1-7/16″	37,520	417	2096	2513
3-3/8″ & 1-7/16″	48,720	541	2722	3263
1-3/8″ & 2-7/16″	41,440	460	2315	2775
1-3/8″ & 3-7/16″	56,560	628	3160	3788
1-3/8″ & 4-7/16″	71,680	796	4004	4800

Sample Calculation: Using 4-7/16″ Strand, e = 8″.

$$\frac{P}{A} + \frac{Pe}{S_b} = \frac{4 \times 15,120}{90} + \frac{4 \times 15,120 \times 8}{143.2} = 672 + 3379 = 4051 \text{ p. s. i.}$$

$$\frac{P}{A} + \frac{Pe}{S_b} = 0, \text{ cancelling, } e = \frac{-S_b}{A} = -\frac{143.2}{90} = -1.591''$$

Therefore, where e = −1.591″, $f_b = 0$.

TABLE 6

f'_{ci}	3000	3200	3400	3600	3800	4000	4200	4400	4600	4800	5000
$\frac{0.60\ f'_{ci}}{1.166}$	1544	1647	1750	1852	1955	2058	2161	2264	2367	2470	2573

$$f_b = 0.60\, f'_{ci} = \frac{P_o}{A} + \frac{P_o e}{S_b} = 1.166 \left(\frac{P}{A} + \frac{Pe}{S_b}\right)$$

$$\frac{P}{A} + \frac{Pe}{S_b} = \frac{0.60\, f'_{ci}}{1.166}$$

TABLE 7

(1) L	(2) L^2	(3) $0.827L^2$	$f'_{ci} = 3000$ Col. (3) + $0.515\ f'_{ci}$	$f'_{ci} = 4000$ Col. (3) + $0.515\ f'_{ci}$	$f'_{ci} = 4500$ Col. (3) + $0.515\ f'_{ci}$
20	400	331	2321	2836	3091
22	484	400	2390	2905	3160
24	576	476	2466	2981	3236
26	676	559	2549	3064	3319
28	784	648	2638	3153	3408
30	900	740	2730	3245	3500
32	1024	846	2836	3351	3608
34	1156	956	2946	3461	3716
36	1296	1070	3060	3575	3830
38	1444	1193	3183	3698	3953
40	1600	1322	3312	3827	4082
42	1764	1459	3449	3964	4219
44	1936	1600	3590	4105	4360
46	2116	1749	3739	4254	4509
48	2304	1905	3895	4410	4665
50	2500	2068	4058	4573	4828
52	2704	2235	4225	4740	4995
54	2916	2410		4915	5170
56	3136	2593		5098	5353
58	3364	2780		5285	5540
60	3600	2975		5480	5735

Assume total of 20% losses.
Assume 1/3 of losses have occurred at transfer.
18,900 — 15, 120 = 3780
3780 ÷ 3 = 1260
18,900 — 1260 = 17640
17,640 ÷ 15,120 = 1.166

$$\left(\frac{P}{A} + \frac{Pe}{S_b}\right) 1.166 - \frac{M_{DL}}{S_b} = 0.60\ f'_{ci}$$

$$\frac{P}{A} + \frac{Pe}{S_b} = \frac{0.60 f'_{ci}}{1.166} + \frac{92L^2 \times 1.5}{143.2 \times 1.166} = 0.515\ f'_{ci} + 0.827L^2$$

3000 x 0.515 = 1990
4000 x 0.515 = 2505
4500 x 0.515 = 2780

Note: Values for e and y should not be used when strand sizes are mixed

6 strands		-.750	-0.75	1.00	2.75	4.50	6.25	8.00	-1.625	-0.750	0.125	1.000	1.875	2.750	3.625	4.500	5.375	
		12.50	10.75	9.00	7.25	5.50	3.75	2.00	11.625	10.750	9.875	9.000	8.125	7.250	6.375	5.500	4.625	
8 strands																		
	12 -	0.125	0.5625	1.000	1.4375	1.8750	2.3125	2.750	3.1875	3.6250	4.0625	4.5000	4.9375	5.3750	1.000	1.350	1.700	
	½"	9.875	9.4375	9.000	8.5625	8.1250	7.6875	7.250	6.8125	6.3750	5.9375	5.5000	5.0625	4.6250	9.000	8.650	8.300	

GRAPH 'A'

Bottom Fibre Stress Due to Dead Load plus Superimposed Load, $f_b = \frac{M_D}{S_c} + \frac{M_{SI}}{S_{cc}}$, lb per sq in

Superimposed Load, lb per sq ft (Roof Members)

* Superimposed Load, lb per sq ft (Floor Members)

*Superimposed load is in addition to 25 lb per sq ft allowed for 2" topping. Based on transformed section for topping having 28 day strength of 4000 psi.

GRAPH 'B'

Bottom Fibre Stress Due to Dead Load plus Superimposed Load, $f_b = \frac{M_D}{S_c} + \frac{M_{SI}}{S_{cc}}$, lb per sq in

Critical de... point

Allowable tension in bottom fibre = 6√f'c = 6√5500 = 44...

Decimal of Span, L

EXAMPLE

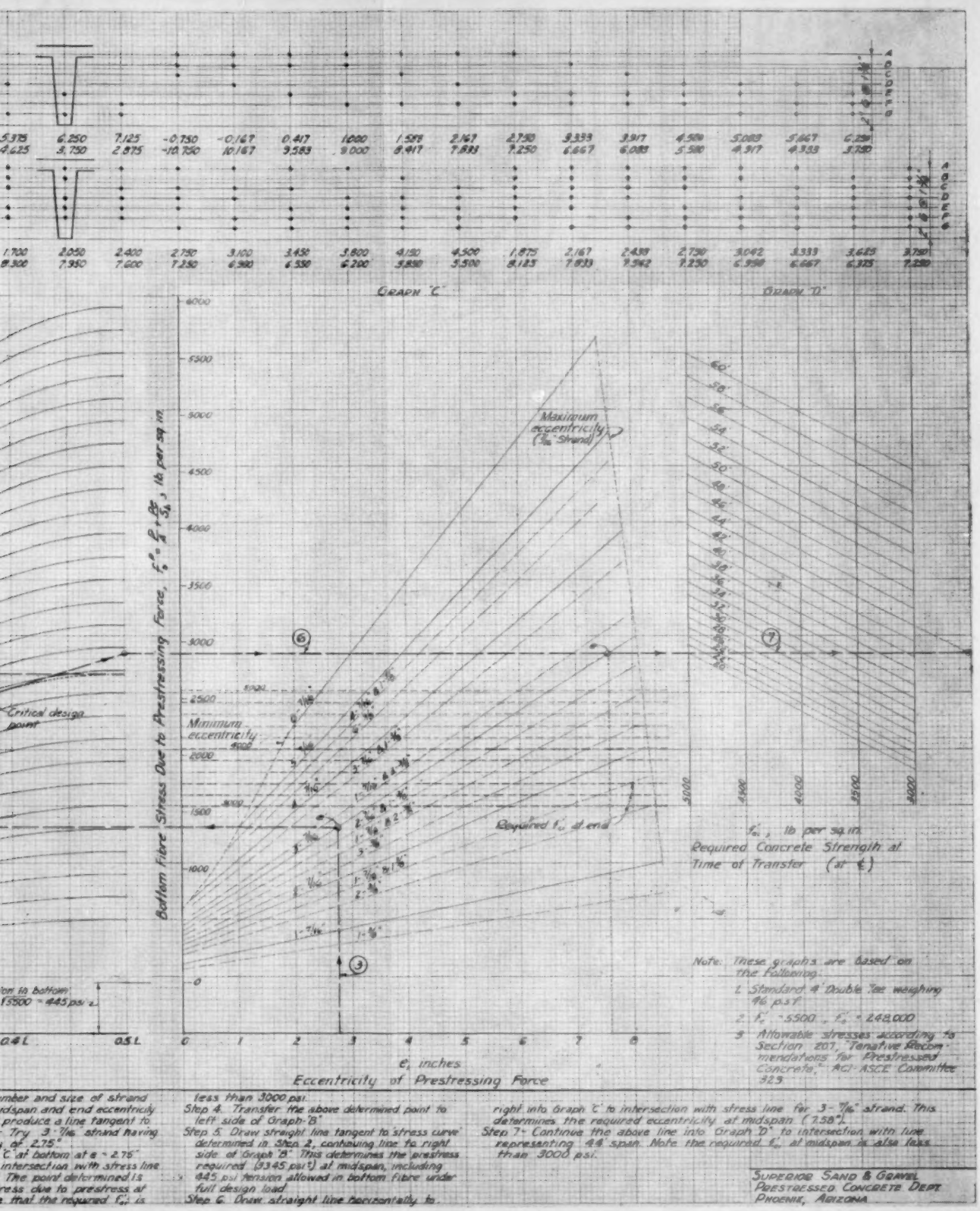
Given: 44' Roof span, superimposed load 32 psf.

Solution:

Step 1. Enter Graph 'A' at bottom at 32 psf. Proceed upward to intersection with line 44' R.

Step 2. From the above determined point go horizontally to right side of Graph 'B' to determine midspan ordinate of the required stress curve. Since the midspan ordinate does not fall on one of the curves shown, by interpolation draw in new stress curve (dotted line). It is now necessary to determine the number and ... required and the midspan and ... of thereof which will produce a ... this stress curve. - Try 3 - ... an end eccentricity of 2.75" ...

Step 3. Enter Graph 'C' at bottom ... Proceed upward to intersection ... for 3 - 7/16" strand. The point ... the bottom fibre stress due to ... end of span. Note that the ...


Graph 'C'
Graph 'D'
Maximum eccentricity (7/16" Strand)
Minimum eccentricity
Required f'_{ci} at end
Critical design point
Bottom Fibre Stress Due to Prestressing Force, $f = \frac{P}{A} + \frac{Pe}{S_b}$, lb per sq in.
e, inches
Eccentricity of Prestressing Force
f'_{ci}, lb per sq in.
Required Concrete Strength at Time of Transfer (at ℄)
0.4 L
0.5 L

Note: These graphs are based on the following:

1. Standard 4' Double Tee weighing 96 p.s.f.
2. f'_c = 5500 , f'_{ci} = 248,000
3. Allowable stresses according to Section 207, "Tentative Recommendations for Prestressed Concrete," ACI-ASCE Committee 323

...mber and size of strand ...idspan and end eccentricity ... produce a line tangent to ... Try 3-7/16" strand having ... of 2.75". ... 'C' at bottom at e = 2.75" ... intersection with stress line ... The point determined is ...ress due to prestress at ... that the required f'_{ci} is less than 3000 psi.

Step 4. Transfer the above determined point to left side of Graph 'B'.

Step 5. Draw straight line tangent to stress curve determined in Step 2, continuing line to right side of Graph 'B'. This determines the prestress required (9345 psi) at midspan, including 445 psi tension allowed in bottom fibre under full design load.

Step 6. Draw straight line horizontally to right into Graph 'C' to intersection with stress line for 3-7/16" strand. This determines the required eccentricity at midspan (7.58").

Step 7. Continue the above line into Graph 'D' to intersection with line representing 44' span. Note the required f'_{ci} at midspan is also less than 3000 psi.

Superior Sand & Gravel
Prestressed Concrete Dept.
Phoenix, Arizona

prestressed concrete saves $400,000 on panama city bridge

By ROBERT N. BRUCE, JR.
Raymond International, Inc., New York, N. Y.

THE RECENTLY COMPLETED new bridge spanning West St. Andrews Bay at Panama City, Florida, will replace the existing parallel structure, the Hathaway Bridge.

A unique feature of this new high level structure is that it marks the first time that the Florida State Road Department has employed the use of hollow precast prestressed concrete cylinders as the main component of the substructure of a bridge. Of the thirty-five bents making up the new substructure, thirty-three rest on prestressed cylinders and the remaining two rest on steel H-pile supported concrete piers which were constructed in open cofferdams.

For the thirty-three prestressed concrete bents, alternate designs were prepared and bid completitively. One alternate consisted of driving 54″ O.D. prestressed cylinders into the Bay bottom, utilizing the cylinders as combination bearing piles and structural columns, and then in twelve bents constructing small piers on top of the cylinders. The other alternate consisted of constructing piers in open cofferdams on steel H-pile foundations for twelve bents and the balance on square prestressed concrete piles. Although all contractors bidding the project submitted their bids based on the prestressed cylinder alternate, it was estimated by the low bidder at the time of the bid letting that approximately $400,000 was saved by the elimination of the cofferdam pier construction. Other contractors estimated the saving as being in the range of $300,000 to $500,000. Considering the substructure contract price of $2,036,973, the economy achieved represents a saving of approximately 20% in the cost of the substructure.

Water depths range up to 35′ deep in the locality of the prestressed cylinder bents which necessitated the use of 54″ O.D. piles having a 5″ wall thickness and a length varying from 80′ up to 104′. These large cylinders were manufactured near New Orleans and transported by barge nearly 300 miles to the job site. Placing of the cylinders was accomplished by jetting and driving with a large single acting steam hammer having a rated striking energy of 48750 foot-pounds per blow. For absorbing this large dynamic force as well as design loads, the large cylinders had all the usual advantage of prestressed concrete plus the added advantage of the large section modulus furnished by the cylindrical shape.

Spans other than the main truss include reinforced concrete spans, prestressed concrete spans, and four continuous unit welded steel plate girder structures.

Channel clearance for the new bridge includes 50′ vertical clearance and 300′ horizontal clearance. The two channel piers were constructed with provisions for future modification which will permit construction of a lift span. The bridge has two 26′ roadways divided by a 4′ median and has 2′ safety curbs on each side.

The new bridge was designed by the Florida State Road Department. The prestressed cylinders were furnished by Raymond International, Inc.

STRUCTURE OF AIRCRAFT SERVICE DEPOT AT GATWICK FOR TRANSAIR LTD.

by

Wilfred John Marshall Haines, A.M.I.C.E.
Director, The London Ferro-Concrete Co. Ltd.

and

Alan James Harris, B.Sc.(Eng.), M.I.C.E.
Partner, A. J. & J. D. Harris, Chartered Civil Engineers

EDITOR'S NOTE:

P.C.I. is grateful to the Institution of Civil Engineers, London, England, for its permission to reproduce this paper. If I were to resume my lecture series, I would use it to bring delight and inspiration to an audience. The paper would be presented as an architect-engineer tour de force comprising creativeness, concrete sculpture, psychology, philosophy — the dramatization of a union of the arts and sciences in building construction. Note the press story released by the Cement and Concrete Association:

> **"The frames are a design triumph — the lace-like lattice of incredibly slender ribs and nodes is an inspiring example of the strength and elegance which can be achieved by a combination of precasting and prestressing."**

Can one resist titillating to that melody? Modestly, they refrain from giving credit to themselves — the high-level architect-engineer minds merged into one. They exclaim poetically about the material. But, it is to the men — not to the material, that I would first direct your attention.

Note the architect's precise and logical selection of type of structure for appearance (his soul at work); concrete shells dismissed (because of cost), steel framing discarded in favor of concrete (to save maintenance, although costs were right), precast prestressed space frames finally selected (for advantages over reinforced concrete).

Then sit back and watch the development of the design, unfold, veritably a gem of analysis and synthesis, simply stated, clearly delineated, "why and how" in its every phase.

Readers inspired to use space frame design would not necessarily adopt the connection details or erection procedure employed on this project. They would naturally vary in different countries. In U. S. A., usual practice would dictate prefabrication of completed units and less field work. But I cannot conceive of any one anywhere manifesting greater intellectualism.

M.P.K.

SYNOPSIS

First discussing the **design peculiarities of prestressed concrete triangulated beams**, the Paper then describes their embodiment in the space-frame secondary beams and the continuous main beams, of 105-ft and 140-ft spans respectively.

In this structure, the layout integrates resistance to horizontal loads with resistance to vertical loads; erection methods were carefully worked out before detailed design was undertaken; very extensive use was made of precasting.

The lessons learned during construction are detailed.

INTRODUCTION

Transair Ltd required a service depot and repair workshops for a variety of aircraft used in private charter. Service accommodation was required for different groups of plants such as three Dakotas and two Viscounts or three Viscounts and two Vikings.

2. Careful arrangement allows five machines to be serviced in an area 280 ft X 110 ft. This is made possible by the use of hydraulic lifts to lower the undercarriages below the floor level, bringing the engines down to man height, thus avoiding the use of service docks and the consequent sterilizing of large areas of floor.

3. The clear height under the roof was established at 30 ft. Sliding folding doors, clear of the centre columns, were incorporated, because they give the complete flexibility necessary with the close packing of aircraft.

4. The construction employed was governed to a certain extent by the fact that this was to be the Company's British Headquarters and, as such, a prestige building, hence, a simple shed or hangar was not considered suitable. Consideration was given by the architects, Messrs Clive Pascall & Peter Watson, to the following constructions:—

(i) Pitched roof in steel and precast concrete	Economic but discarded because of appearance
(ii) Arched form in precast concrete	Economic but discarded because of appearance
(iii) Thin concrete shells of various outlines	Discarded because of cost
(iv) Steel lattice girders and secondary framing giving flat roof	Economic but discarded in favor of concrete to save maintenance
(v) Precast concrete triangulated beams	First cost as steel and added advantage of reduced maintenance

The building capacity was a marginal million cubic feet and concrete was favoured by the fire-prevention authorities. In consequence, scheme (v) was chosen.

Peculiarities of Triangulated Prestressed Concrete Beams in General

5. The choice of a triangulated beam rather than a solid-web beam is dictated by the same circumstances when the beam is of prestressed concrete as when it is of steel; i.e. large depth available and/or light loads.

6. The advantages are also similar: depth can be increased and members lightened without the penalty of wasting material in a web which is unnecessarily strong; deflections can be reduced; erection may be simplified. (Prestressed concrete has an additional advantage in that it is easier to take economic advantage of very-high-strength concrete, but this probably applies only to very large structures.)

7. The design of intersections is clearly critical, and it is prudent to ensure that the concrete in intersections is always submitted to compressive forces in equilibrium. Minor binding reinforcement would then seem to be adequate.

8. The question of secondary stresses requires discussion. The gravity of exaggerated secondary stresses in metal structures lies more in the dangers of shear failure in jointing devices and fatigue failure owing to repeated loading than in simple overstressing. With prestressed concrete, the situation seems slightly different:—

(a) Secondary stresses will exaggerate either the compressive or tensile stresses in the concrete; the tendons being generally axial, they will be little affected.

(b) With increased compressive stresses, there will be increased local creep which will automatically relieve the stresses. It is conceivable that the application of load may be so large and so sudden as to produce a local compression failure before non-elastic strain has occurred, but in a large structure this seems unlikely.

(c) With increased tensile stresses, cracks open, and a semi-articulation is automatically created.

(d) A concrete hinge is much more easily constituted than a metal hinge; the fitting of virtual pin joints at the end of each member is conceivable.

9. The problem of buckling is particularly pertinent to triangulated structures, since ignorance of the real resistance to buckling can lead to an exaggeration of the size of compression members, which may lose much advantage in dead weight.

10. The essence of the problem is the choice of a value of the modulus of elasticity. Now with permanent loads, it is known that this modulus can reach much smaller values than with transient loads, but when a transient load is superimposed on a permanent load which has been acting for, say, a period of years, it is impossible at the moment to predict the behaviour. Caution is therefore obligatory.

11. It is worth noting that such structures erected in Russia and Eastern Europe incorporate compression members of extreme slenderness, e.g. l/b of 40 to 50. The use of such slenderness ratios seems common and it must be assumed that they are found satisfactory.

Bases of Design

12. The following values were used for design:—

Loads

15 lb/sq. ft. plus wind-load exposure C (p = 12 lb/sq. ft.).
Hoist applying 2-ton load on bottom of secondaries.

Working stresses

(a) Concrete: 6,000 lb/sq. in. minimum strength at 28 days.

Maximum bending compression	2,000 lb/sq. in.
Maximum direct compression	1,500 "
Maximum prestress compression	2,000 "
Maximum tension	zero.

Increase of 33 1/3% under wind load on all concrete stresses.

(b) Steel in tendons: 95-105 tons/sq. in. ultimate tensile strength.

Maximum tension after relaxation	60 tons/sq. in.

Load factor: 1.5 times dead plus 2.5 times live load, or twice design load, whichever is the greater.

Layout of Structure

13. The roof was supported on three main columns in the front and columns at 20-ft centres around the sides and back (Figs 1a and 2). These latter were completely independent of the columns supporting the annexes to permit free thermal movement of the roof. The combination of main beam, triangulation in the plane of the roof, and transverse members gave the roof structure adequate stiffness and strength to carry wind loads back to the three front columns and a braced panel in the back wall. The centre column was made rigid in all directions, the wing columns were made rigid in the short-span direction and flexible in the other, and the braced panel was made rigid in the long-span direction and flexible in the other; all other columns were adequately flexible in all directions. The structure was thus stable under wind forces and yet able to expand and contract freely.

14. The three main columns were cast in situ and prestressed; all the secondary columns were precast and reinforced. The heads of the side columns were attached to the beam ends by flexible ties permitting vertical movement of the beams; these side columns were thus vertical beams rather than compression members.

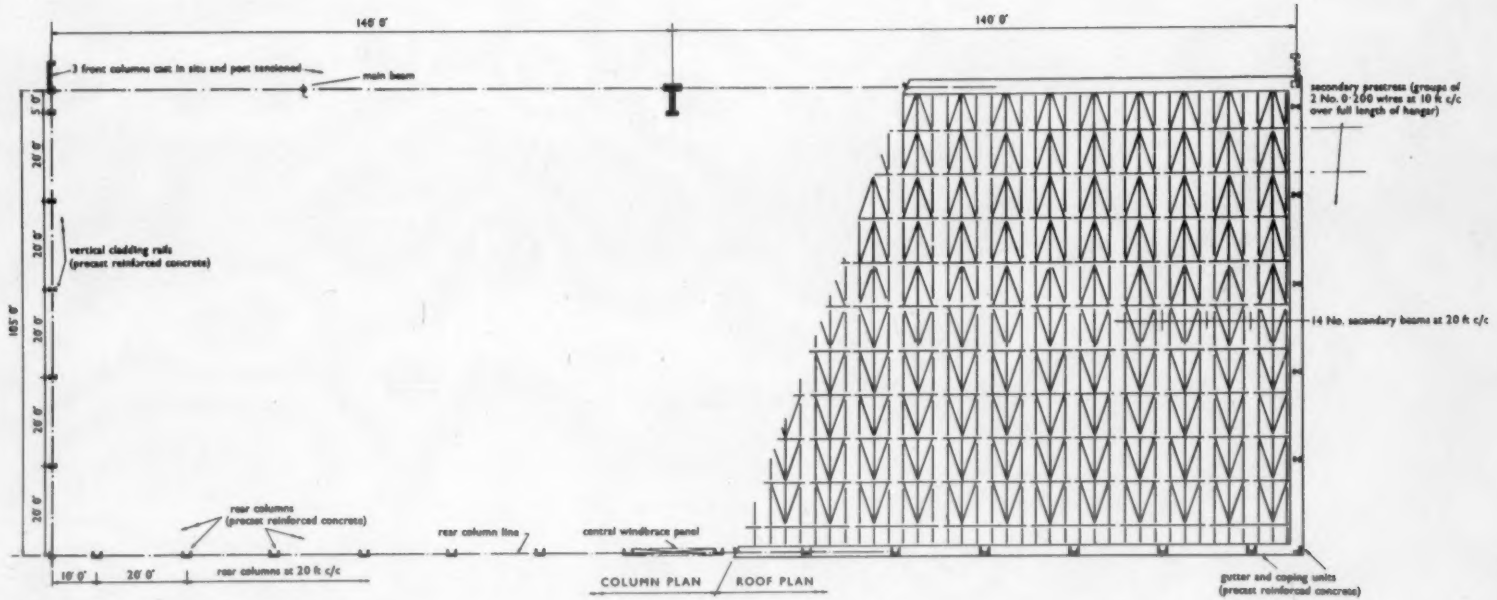

140' 0'
140' 0'
3 front columns cast in situ and post tensioned
main beam
secondary prestress (groups of 2 No. 0·200 wires at 10 ft c/c over full length of hanger)
vertical cladding rails (precast reinforced concrete)
14 No. secondary beams at 20 ft c/c
185' 0'
5' 0'
20' 0'
20' 0'
20' 0'
20' 0'
20' 0'
rear columns (precast reinforced concrete)
rear column line
central windbrace panel
10' 0'
20' 0'
rear columns at 20 ft c/c
COLUMN PLAN
ROOF PLAN
gutter and coping units (precast reinforced concrete)

Fig. 1a. General Layout of Structure

Fig. 2. General view of structure

Secondary beams

15. The general arrangement is shown in Figs 1a and 1b.

16. The triangulation in the upper surface served three purposes:—

(a) It provided torsional stiffness.

(b) It provided stiffness in the deck under horizontal loads.

(c) The longitudinal forces applied by the diagonals were spread between the longitudinal members, which could thus act not only as cladding bearers but as the compression boom as well. These bearers were reinforced to carry bending under zero compression.

The cladding was of Stramit panels 3 ft 4 in. broad with bituminous felt on top.

17. The prestressing arrangements consisted of 2 X 12/276 Freyssinet cables lying in open grooves in the bottom boom, and a number of single 276 wires ranging from four to one lying in holes in the tension diagonals.

Main beams

18. The important features are the following:—

(a) The monolithic union of main and secondary beams, resulting from building the end frames of the latter into the former. All risk of torsional buckling was thus avoided; in addition, torsional loads caused by wind on the doors were supported by bending in the secondary beams.

(b) Placing the main tendons in a parabolic profile on either side of the diagonal members. This counterbalanced a large part of the shear and greatly reduced the forces in the diagonals and the number of stressing operations; in fact, all but one or two of the diagonal tendons lay in the secondary-beam end-frames and could be tensioned prior to erection.

Fig. 3. Symmetrical pattern of beams

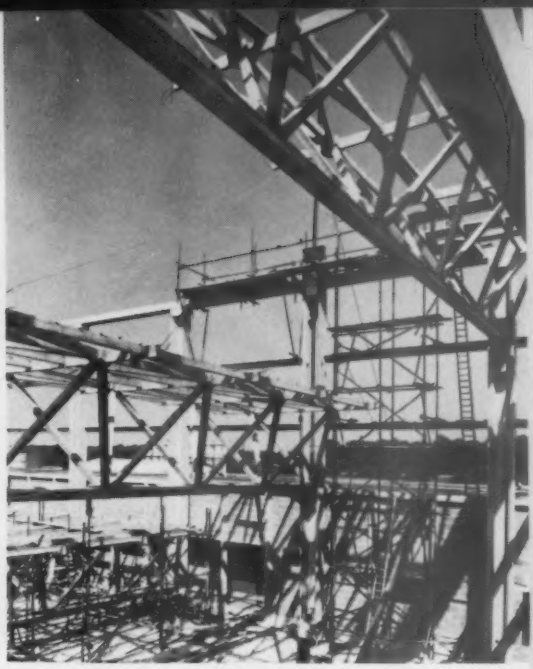


Fig. 4. Hoisting of secondary beams

19. The beam was analysed as a hyperstatic triangulated beam; the approximation, consisting of replacing it by a continuous solid-web member of constant moment of inertia, was employed only for preliminary design.

20. The design was thus as follows:—

(i) Approximate analysis to establish sizes of members and general feasibility of details.

(ii) "Exact" analysis of preliminary beam under the following loads:

(a) Unit loads at top panel points.

(b) Unit loads at bottom panel points.

(c) Unit compressive load along top boom.

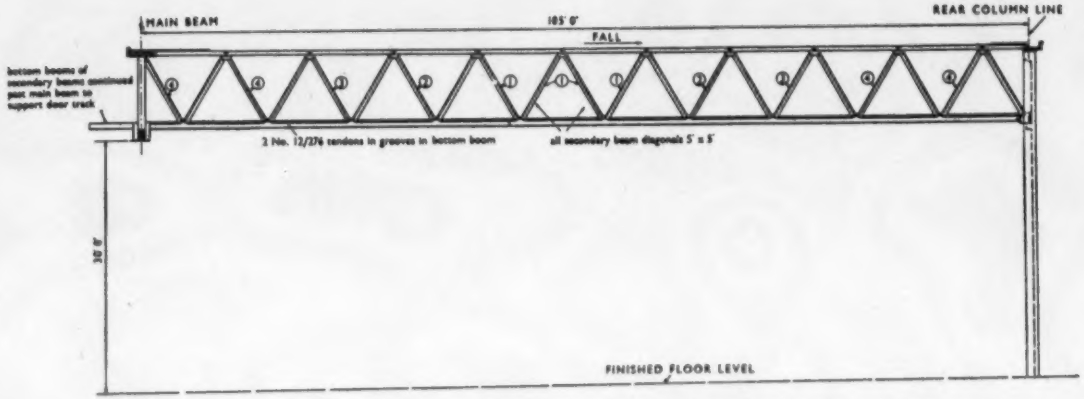
(d) Unit compressive load along bottom boom.

(iii) From these basic figures all combinations of load and prestress could be built up; e.g. the effect of prestress alone was the sum of different multiples of (b), (c), and (d).

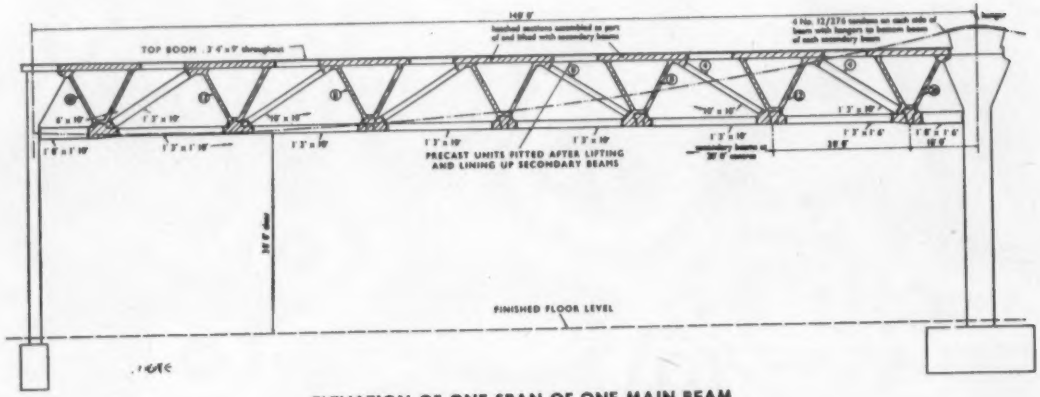
(iv) Operations (ii) and (iii) were repeated after modifications under (iii) to give the final design.

21. The resulting arrangement of prestress has imposed an interesting symmetry on basically unsymmetrical beams (Fig. 3). Table 1 gives the forces and stresses in critical members under different states of load.

22. Some apprehension was felt as to whether the hangers would remain vertical, when the main tendons were stressed; the hanger heads were accordingly connected together by mild-steel tie-rods designed to support the fricton forces resulting from the tension of one cable in each group; this cable would itself lock the hanger against displacement caused by tensioning subsequent cables. Each cable was designed to be overtensioned and then released slightly to establish a reasonably uniform tension in each group of tendons. The tendons were cased in a split pitch-fibre pipe and then filled with grout; there is thus a double protection against corrosion.


MAIN BEAM
185' 0"
REAR COLUMN LINE
FALL
bottom booms of secondary beams continued past main beam to support door track
2 No. 12/276 tendons in grooves in bottom boom
all secondary beam diagonals 5' x 5'
FINISHED FLOOR LEVEL

Cross- section of hangar showing side elevation of secondary beam



ELEVATION OF ONE SPAN OF ONE MAIN BEAM

Fig. 1b. GENERAL ARRANGEMENT OF BEAMS

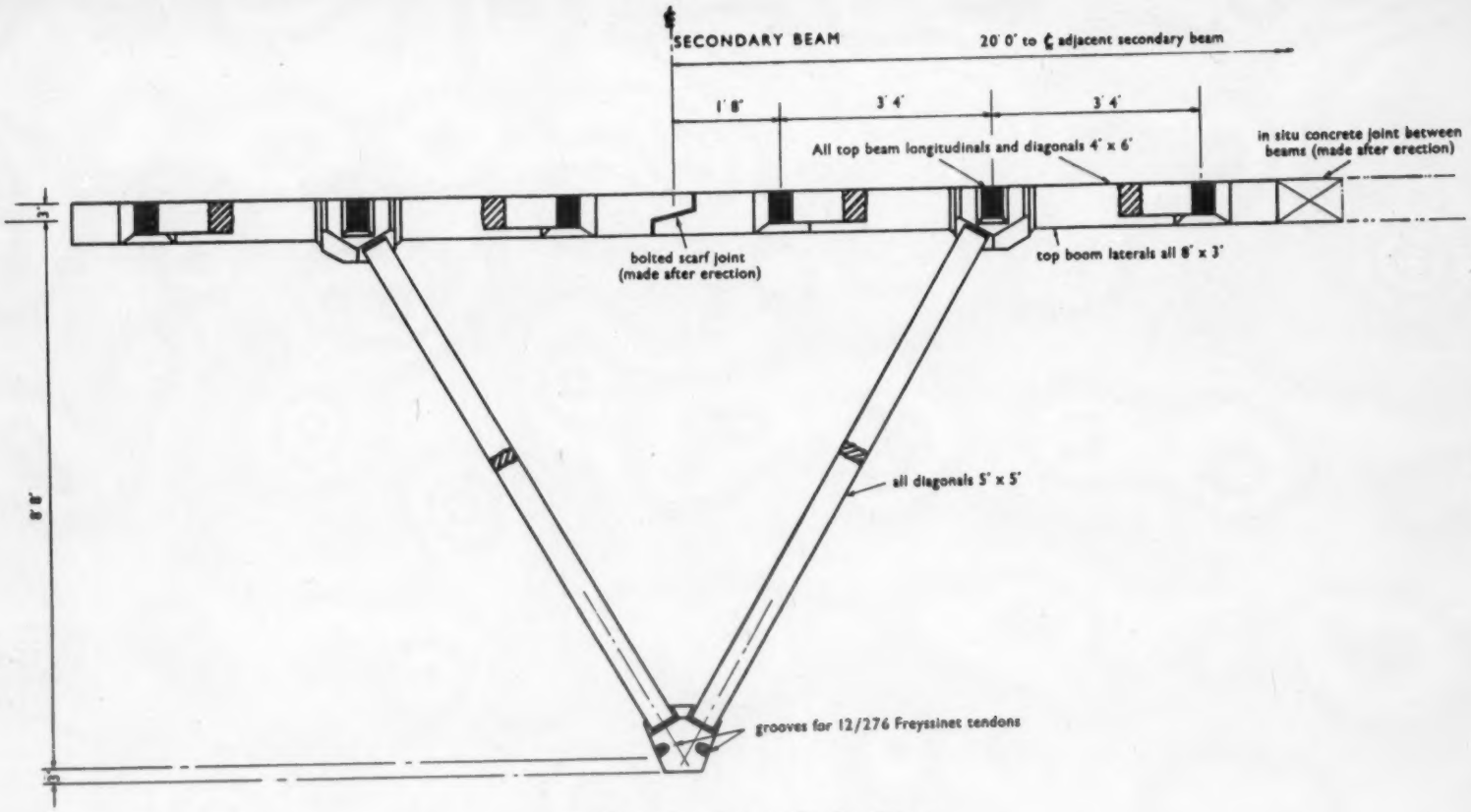

SECONDARY BEAM
20' 0" to ℄ adjacent secondary beam
1' 8"
3' 4"
3' 4"
All top beam longitudinals and diagonals 4" x 6"
in situ concrete joint between beams (made after erection)
bolted scarf joint (made after erection)
top boom laterals all 8" x 3"
all diagonals 5" x 5"
grooves for 12/276 Freyssinet tendons

Fig. 1c. Cross-section of secondary beam

23. It will be noted that certain diagonals were stressed after constitution of the beams; as noted above, the effect of such stressing is complex. In fact, this effect was estimated to be small and the complexity was ignored.

24. The parabolic tendons consisted of two groups of 4 X 12/276 Freyssinet cables each; the diagonals were stressed by various numbers of 4/276 prestressing tendons.

Erection

25. Erection was regarded as an intrinsic part of the design problem; the method of erection was in fact settled before detailing of the structure was undertaken. The erection sequence was:—

(i) In-situ construction of main columns.

(ii) Precasting and placing of secondary columns and fixing of gutters; the latter had a certain stiffening effect.

(iii) Construction of secondary beams on the ground by assembly of precast elements on scaffolding.

(iv) Stressing of secondary beams.

(v) Hoisting of secondary beams into position by jacking on two steel straps from scaffolding towers in front and one steel strap lying in the hollow of the column in the rear (see Fig. 4). The movement of the secondary beam was purely vertical.

(vi) Constitution of main beam by placing precast elements between secondary-beam end frames.

(vii) Stressing of longitudinal roof.

(viii) Decking of roof.

(ix) Stressing of main cables.

(x) Cladding, fixing of doors, etc.

26. The following features of design were considered helpful to erection:—

(a) The shaping of the rear columns to permit the secondary-beam end blocks to slide vertically along them.

(b) The small quantity of work needed on the roof deck once the secondary beams were in position; these beams constitutes virtually the complete deck.

(c) The employment of parabolic main tendons in the main beam with the consequent reduction of diagonal stressing.

(d) The use of secondary-beam end frames as part of the main beam simplified both the placing of the secondaries and the construction of the main beams.

27. The following **features were a nuisance** and would be avoided or improved if possible on a future occasion:—

(a) Stressing of the transverse members. This was done by wires in open grooves; the inevitable variation of level of the beams was nearly as large as the depth of the groove. It is considered that the in-situ reinforced concrete joint could have been made adequate; this would also have avoided one or two complicated anchorage details.

(b) Threading the main cables to the main beam through individual holes in the hangers was time-consuming; an attempt would be made to have a hanger detail open at the side.

(c) The number of minor variations on the standard roof A-frames could have been reduced by modifications of layout.

(d) The main-beam unit forming the front end of the secondary beam was detailed as a V-shape incorporating a very short section of the main-beam bottom boom. This short bottom-boom section was supported at its ends on the front scaffolding and the necessarily close centres of the supports rendered it difficult to adjust the secondary beam to a proper level by jacking off the scaffold. A longer bottom section spreading the end supports out to, say 7 ft, would have made small final adjustments more simple.

(e) It was found necessary to break the steel lifting straps into two lengths to avoid an unmanageable length of flexible strip projecting above the hangar roof when

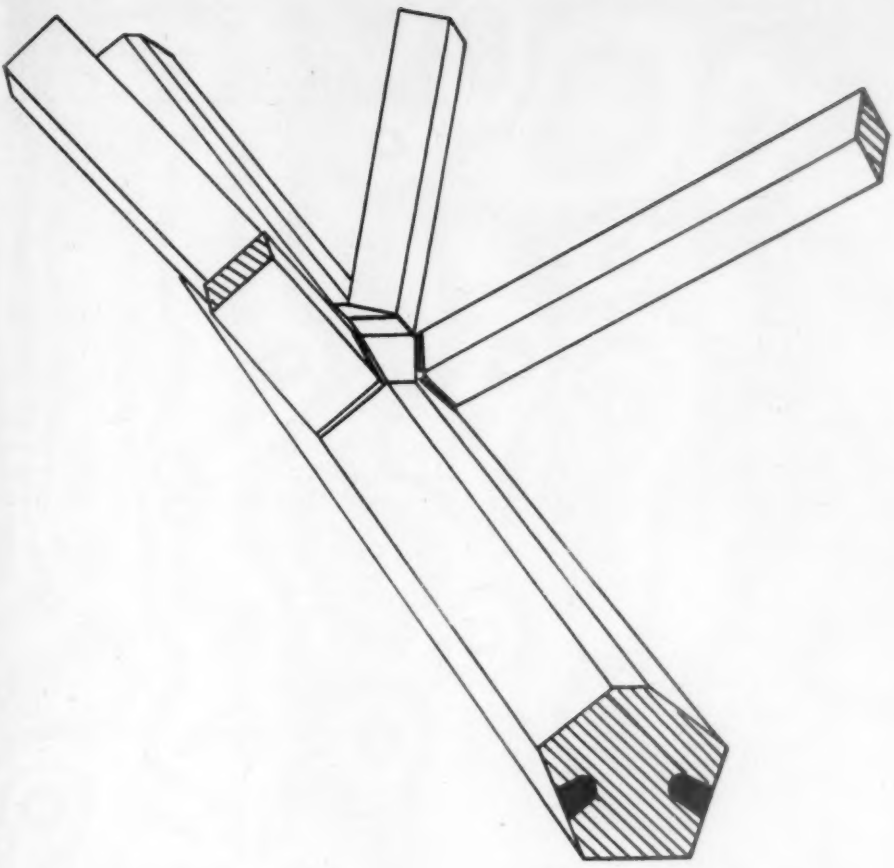


Fig. 1d. Secondary beam—detail of junction of diagonals and bottom boom

when hoisting was complete. The dividing of these straps meant supporting the secondary beam in a half-way position. Extra holes through the flanges of the rear columns would have made it simple to support the beam on a short length of steel joist.

28. In general, erection proceeded smoothly and was free of incident or sensation.

Other details

29. The cladding consisted of "Plyglass" panels, which consist of a sandwich of two sheets of glass with fibre-glass between; these panels diffuse light and have a certain thermal-insulation value. The panels were fixed in aluminum glazing bars supported on reinforced concrete rails bolted to the columns.

30. The canopy over the doors was finished with an aluminum fascia supported on a light steel girder bolted to the projecting ends of the secondary-beam booms.

31. Lighting fitments were fixed to the secondary-beam booms by light metal clips, and this operation was carried out while the beams were at ground level.

TABLE 1.—FORCES IN CRITICAL MEMBERS

Type of load	Point of application	Magnitude	Top boom		Bottom boom		Diagonals	
			X9	X23	D11	H22	21.22	19.20
Dead load	Top panel points	21·65	+ 299·4	− 578·0	− 282·8	+ 478·8	− 191·0	+ 275·3
„ „	Bottom panels points	10·9	+ 78·0	− 146·2	− 71·4	+ 121·8	− 54·5	+ 69·5
Live load −ve	Top panel points	−7·56	− 103·0	+ 199·5	+ 97·0	− 161·0	+ 65·8	− 95·7
Live load +ve	Top panel points	+8·0	+ 111·0	− 213·0	− 105·0	+ 178·0	− 71·0	+ 99·5
Total force minimum: thousands of lb.			+ 274·4	− 524·7	− 257·2	+ 435·6	− 179·7	+ 249·2
„ „ permanent: „			+ 377·4	− 724·2	− 354·2	+ 600·6	− 245·5	+ 344·8
„ „ maximum: „			+ 498·4	− 937·2	− 459·2	+ 778·6	− 316·5	+ 444·3
Prestress uplift	Bottom panel points	24·85	− 177·7	+ 333·2	+ 162·0	− 276·1	+ 123·7	− 157·5
Prestress thrust	Bottom boom	642	+ 232·0	+ 648·0	+ 340·5	+ 16·1	+ 50·6	− 82·8
Secondary prestress	In member		—	—	—	—	+ 150·0	+ 30·0
Prestress total			+ 54·3	+ 981·2	+ 502·5	− 260·0	+ 324·3	− 210·3
Resultant force minimum: thousands of lb.			+ 328·7	+ 456·5	+ 245·3	+ 175·6	+ 144·6	+ 38·9
„ „ permanent: „			+ 431·7	+ 257·0	+ 148·3	+ 340·6	+ 78·8	+ 134·5
„ „ maximum „			+ 552·7	+ 44·0	+ 43·3	+ 518·6	+ 7·8	+ 234·0
Cross sectional area: sq. in.			360	360	150	360	60	150
Resultant stress minimum: lb/sq. in.			+ 912	+1,270	+1,635	+ 488	+2,410	+ 259
„ „ permanent: „			+1,200	+ 715	+ 990	+ 948	+1,313	+ 897
„ „ maximum: „			+1,530	+ 122	+ 248	+1,440	+ 130	+1,560

Strain and Deflection Measurements

32. In order to assess the performance of the structure, readings were taken of strain-deflection in association with the Cement & Concrete Association, to check the following:—

(i) Deflection of main beam.

(ii) Differential settlement of main columns.

(iii) Deflection of secondary beams.

(iv) Strains in diagonals of secondary beams.

33. The mid-span deflection of each half of the main beam as a result of tensioning was between ⅛ in. ¼ in. upwards; the small size of this deflection is due to the nearly equal mean stresses in the top and bottom booms under deadload.

34. Owing to lack of time, strain measurements had to be confined to the secondary beams, two of which were tested. Strain readings were taken on all four sides of selected diagonals at the ends and near the midle of the beams. For each case the mean stresses on the middle diagonals were similar to those given by a pin-joint analysis but at the ends the actual strains were lower, largely because of the stiffness of the end joints and of the the bottom boom. The strain readings showed quite appreciable bending in the ends of the diagonals although virtuallv no tensions were recorded; it is anticipated that creep will "even-out" these strains to some extent. Further readings of strain and deflection are being taken to assess the effects of time.

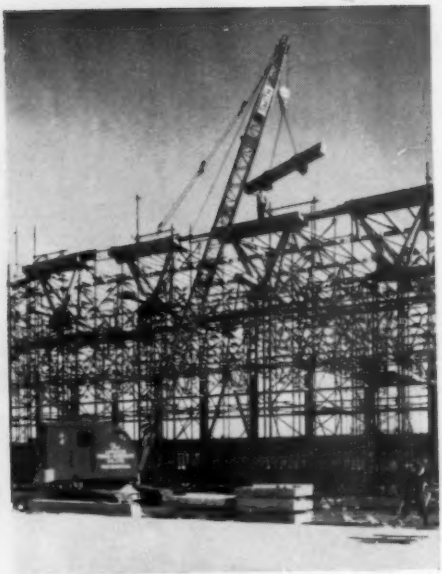
Acknowledgments

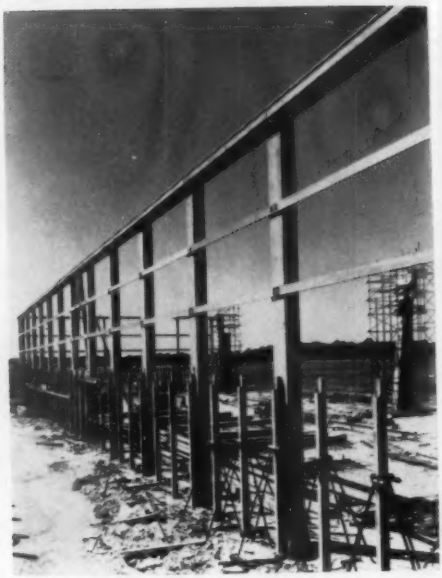
35. The architects were Messrs Clive Pascall & Peter Watson. The main contractors were Sir Alfred McAlpine & Son Ltd. The subcontractors for the concrete structure were The London Ferro-Concrete Co. Ltd.

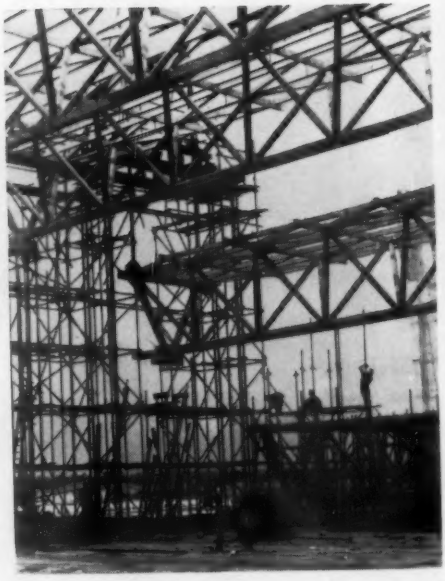
36. The design was drawn up by The London Ferro-Concrete Co. Ltd and Messrs A. J. & J. D. Harris, Chartered Civil Engineers, in collaboration. Mr. R. H. C. Galbraith, M.C., B.E., A.M.I.C.E., of Dunedin, New Zealand, who was in England at the time, gave assistance in the analysis of the structure.

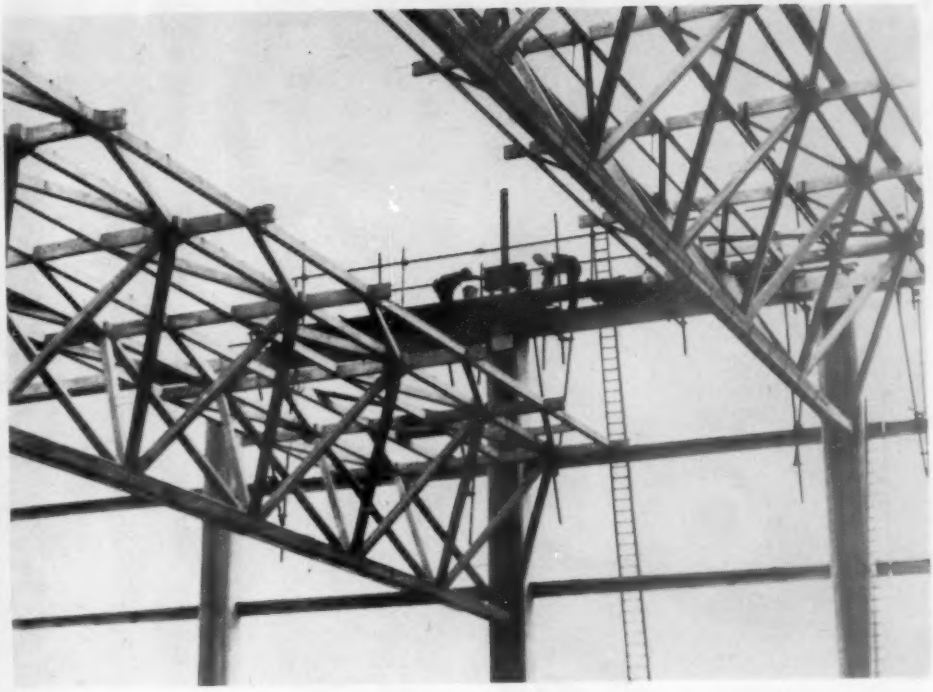
Following are photographs

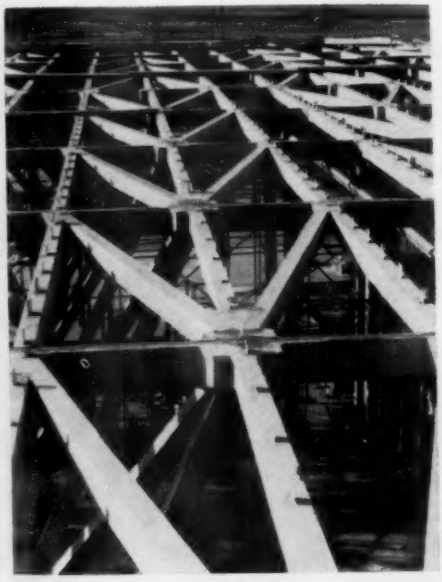
showing various stages of construction











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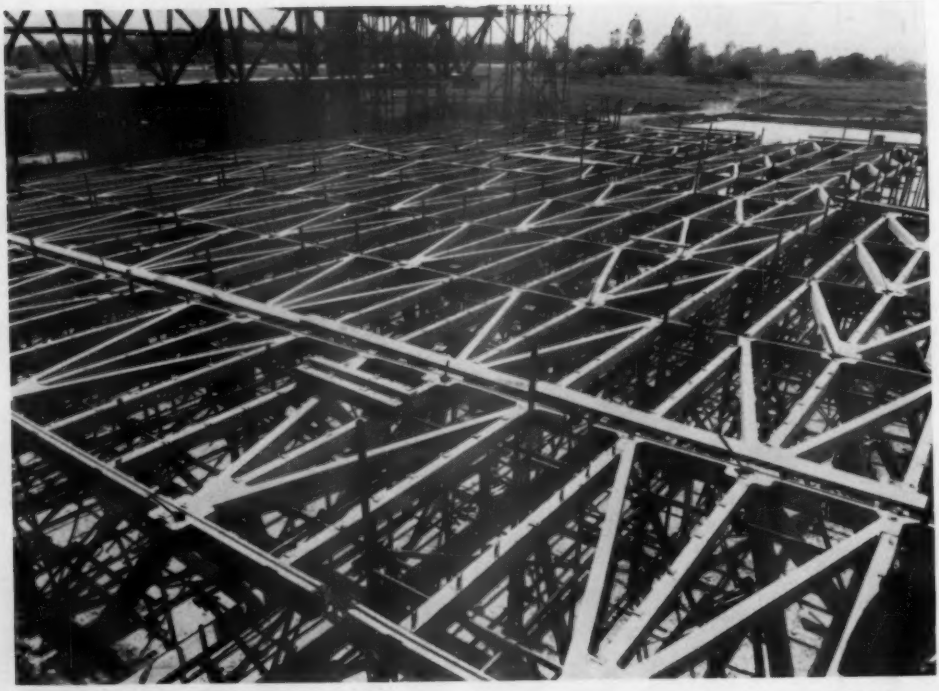

A Report on Differential Shrinkage in Composite Prestressed Concrete Beams

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FLORIDA STATE ROAD DEPARTMENT

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INTRODUCTION

The purpose of this investigation was to study, both experimentally and analytically, the effects of shrinkage and creep in composite construction consisting of a prestressed-concrete beam and cast-in-place slab (Fig. 3). The effect of shrinkage and creep of concrete and the relaxation of steel stress in ordinary prestressed beams is known to be a major cause of loss of prestress, but little is known about the magnitude of the induced stresses resulting from differential shrinkage of the composite beam.

The term differential shrinkage is used here for the effect of the relative strains in the two concretes caused by the shrinkage of the cast-in-place concrete and shrinkage plus creep of the prestressed concrete when the two are bonded together. The free-shrinkage strain in the cast-in-place slab (shrinkage without restraint from other bodies) exceeds the shrinkage-plus-creep strain in the prestressed stem. When the two are bonded together, this has the effect of inducing a compressive force at the top of the beam and a tensile force at the bottom of the slab. If the slab is reinforced, there is an additional concrete stress caused by the resistance of the reinforcing steel to shortening. This stress is tensile in all cases except where the slab steel is outside the kern of the slab section, in which case a compressive stress is induced on the opposite extreme fiber.

Differential shrinkage may cause stresses of considerable magnitude in the prestressed portion of the composite beam. Actually, in certain cross sections in which the cast-in-place part is relatively massive and the prestressed portion, slender and deep, it is possible for these stresses to be as high as those caused by live load. Hence the stresses resulting from differential shrinkage in any given design should be taken into account.

The analysis of shrinkage and creep in composite prestressed-concrete beams can be divided into three parts:

1. The loss of prestress because of shrinkage and creep of the prestressed concrete and relaxation of steel stress as in ordinary prestressed-concrete beams
2. The fiber stresses in both portions of the composite section resulting from differential shrinkage.
3. The additional tensile stresses in the slab, if it is reinforced, caused by the ordinary shrinkage of a reinforced-concrete element.

The ordinary shrinkage and creep of the beam and the shrinkage stress in the slab, if reinforced, are separate effects which must be considered in addition to the differential shrinkage effect. If the slab were not reinforced and there were no shrinkage or creep in the beam, the differential effect would still occur because of the shrinkage or shortening of the slab concrete.

PURPOSE AND SCOPE

The most important variables influencing the effect of shrinkage and creep in composite prestressed-concrete beams are the following:

1. Relative casting dates
2. Surrounding moisture conditions
3. Whether slab is reinforced or not
4. Relative quality of the two concrete mixes
5. Magnitude of prestress.

The purpose of this investigation was to supply the necessary information for one combination of these variables. Additional tests will be necessary to evaluate the relative influence of each of these variables.

The experimental data obtained in this investigation include the following:

1. Free-shrinkage strain readings on two shrinkage specimens of the high-strength concrete used in the prestressed beams
2. Strain readings on one noncomposite beam and on the prestressed portion of one composite section
3. Strain readings on the slab portion of the composite section
4. The magnitude of induced differential-shrinkage force applied to both portions of the composite beam by the resultant shrinkage of the cast-in-place slab.

A simple procedure based on an experimentally determined coefficient was developed for calculating the differential-shrinkage force and the corresponding fiber stresses in both portions of the composite beam. The magnitude of the calculated force was verified directly from test data.

As a preface to the differential-shrinkage consideration, the results are presented for the free shrinkage of high-strength concrete and the shrinkage plus creep for the noncomposite prestressed concrete beam. The strains of the shrinkage specimens were subtracted from the strains recorded on the beam, and the remaining strain was assumed to be the result of concrete creep. Since the relaxation of steel stress is essentially a stress reduction at constant length, this effect was not included in the measured beam strains.

DESCRIPTION OF SPECIMENS AND TEST PROCEDURE

In these tests two beams 10 in. wide, 14 in. deep, and 24 ft. long and two shrinkage specimens having the same width and depth were used. All four of these elements were cast at the same time. Both beams were post-tensioned at 12 days with four ¾-inch straight bars and an initial prestress force of 52,000 lb. per bar. A reinforced concrete flange 4 in. deep and 36 in. wide was poured on one beam at 104 days. The contact surface was roughened and wet down before the slab was poured, and ½-inch stirrups, spaced at 1 ft. on centers, were used to tie the slab to the beam. Both concretes were vibrated mechanically. The specimens are shown in Figures 1 and 2 and their details, including location of gages, in Figure 3.

The two concrete mix designs per cubic yard of concrete were as follows:

High strength concrete (6060 psi strength at 28 days):

Cement	658 lb.
Sand	1095 lb.
Stone	1716 lb.
Pozzolith	1.75 lb.
Water	38.6 gal.

Slab concrete (4075 psi strength at 28 days):

Cement	451 lb.
Sand	1343 lb.
Stone	1795 lb.
Pozzolith	1.25 lb.
Water	32.0 gal.

Concrete strains were measured with a Whittemore mechanical gage having a 10-in. gage length. Gage points were steel inserts imbedded in the concrete. (Electrical gages were also used in an effort to corroborate the mechanical gage readings, but after a short period of time they showed excessive leakage.) Four mechanical gages were used on one side of each beam, four on each side of the two shrinkage specimens, and nine on the slab. Thus a total of 16 mechanical gages were used on the two shrinkage specimens, 13 on the composite beam and 4 on the noncomposite (bare) beam. The gages on both beams were placed at mid-span.

In order to determine experimentally the magnitude of the resultant force applied by the cast-in-place slab on the prestressed portion of the composite beam as a result of differential shrinkage, one of the prestressed beams was calibrated by applying varying forces along its top surface while taking corresponding beam strain readings at mid-span. For the purpose of calibration, a tensioning bar was placed in a recess in the top of the beam and then removed before the casting of the slab. Using this calibration it was possible to determine the differential shrinkage force directly by a comparison of the strains of the noncomposite and composite sections. The difference in these two strains at a particular gage location was the beam strain resulting only from differential shrinkage.

Additional tests were conducted to throw some light on the expansion and shrinkage of concrete beginning immediately after casting. The results are discussed in the appendix.

TEST RESULTS

Shrinkage Specimens

On each side of the two shrinkage specimens 4 mechanical gages were placed in locations corresponding to those on the stem of the composite beam, as shown in Figure 3. All gage readings were consistent and no significant difference was found between the srinkage-strain readings at gages 1, 2, 3 and 4. One mean value of the shrinkage strains recorded at each of the 16 gages is shown in Figure 4a. This can be considered as the uniform shrinkage strain over the cross section.

The strains of the shrinkage specimens were recorded beginning with the seventh day after casting. The leveling off of the curve indicates an ultimate shrinkage strain of about 4×10^{-4} in./in. Test results presented in the appendix indicate that this value may be increased about 15 per cent when the early shrinkage is included in the measured strain. Laboratory shrinkage is expected to be higher than field shrinkage because of the dryer air in the laboratory. Professor Kluge (1) reported shrinkage strains on some identical specimens to be 5×10^{-4} in./in. in the laboratory and 2×10^{-4} in./in. in the field.

Noncomposite Beam

The strain readings on the noncomposite prestressed concrete beam were recorded from the day of post-tensioning (12th day). Four gages were placed from top to bottom on one side of the beam at mid-span (Fig. 3). The accumulated strains resulting from shrinkage and creep of the concrete were recorded at the four gages. These results are shown in Figure 4b.

It was assumed that by subtracting the shrinkage strain which occurred after the beam was post-tensioned from the combined shrinkage and creep strain, the concrete-creep strain would be obtained. The creep curves are shown in Figure 4c for each gage location, and the elastic-and creep-strain distributions are shown in Figure 5. The creep-distribution values show that, under the stress distribution resulting from prestress and dead load (usually in the neighborhood of the allowable compressive stress at the bottom fiber), creep is not proportional to stress in the lower portion of the beam where the stress is higher. In this stress range the concrete stress-strain curve is approximately straight, so that the elastic-strain distribution-curve values in Figure 5, when multiplied by the modulus of elasticity of the concrete, give the stress-distribution curve. Here, as in succeeding parts of this report, the term "estimated ultimate strain" refers to the estimation of the asymptotes to the different curves from the test data. It is felt that the period of observation was long enough to permit this to be done satisfactorily.

The estimated ultimate creep-strain distribution is shown to vary linearly to about mid-depth in Figure 5. The corresponding beam stress at this point is about $((70 + 810)/2)\ (10^{-6})\ (4.1 \times 10^{-6} = 1{,}800$ psi. Hence for this case, the creep strain is proportional to compressive strain (or stress) up to about 0.3 of the ultimate strength of the concrete.

The estimated ultimate loss of prestress because of concrete shrinkage and creep is about 24.2 percent. This value is higher than is usually assumed but is believed to be correct in this case under laboratory conditions. The creep ratio (ratio of ultimate concrete creep to elastic strain) was found to be about 1.1. (The ACI-ASCE Joint Committee recommends a creep ratio of 1.0 to 3.0.)

Composite Beam

The effect of differential shrinkage is to induce stresses in both portions of the composite section as a result of shrinkage and creep that exists only if the two sections are bonded together. For this investigation one composite and one noncomposite beam were studied so that the difference in the strains of the beam portion of the composite section and the noncomposite beam, each measured from the time of slab casting, give the resulting differential-shrinkage strains (Fig. 7).

The strain readings in the bottom of the slab were used to check the top-fiber beam strains since the two should be the same.

(1) Kluge, Ralph W., "Field Studies of Prestress Loss Due to Shrinkage and Creep," Journal of Prestressed Concrete Institute, June, 1958.

The average strain in the bottom of the slab (Fig. 6) was found to be about 20 percent less than the top-fiber strain in the beam. This can be explained by the fact that there was a three-day delay after the slab cast before the gages could be mounted on its bottom. The gages on the top of the slab were used only as a check and showed larger compressive values than those on the bottom because of the positive (downward) bending of the slab. The gages toward the edges of the slab showed about 15 percent larger compressive strains than those near the stem. For the purposes of this study, only the average of the gage readings on the bottom of the slab are reported.

The differential-shrinkage force was calculated using a coefficient obtained from the differential-shrinkage strain distribution determined experimentally (Fig. 7). This force was verified directly using this same strain distribution along with the calibration that was made between the composite-beam gage readings and an applied longitudinal force at the top of the beam before the slab was poured.

DEVELOPMENT OF THE THEORY OF DIFFERENTIAL SHRINKAGE

It is necessary to understand the nature of the stresses and strains occurring in both portions of the composite section, as a result of concrete shrinkage and creep, in order to explain the calculations needed for determining the differential-shrinkage force and the corresponding fiber stresses. As the slab tends to shorten, relative to the stem of the composite section, a longitudinal compressive force is applied at the top of the stem. The resistance of the stem to slab shortening produces an equal tensile force on the bottom of the slab. The total effect of the slab shrinkage is to cause downward bending of the composite beam and a strain distribution such as shown in Figure 8c. Supplementary tests (results in the appendix) indicated that the bond and shrinkage of fresh concrete are sufficient to begin differential-shrinkage action about 24 hours after casting. Prior to this there is an expansion of the slab resulting from heat of hydration of the concrete and subsequent contraction upon cooling, so that at the end of 24 hours following slab casting, the net slab shortening begins to occur.

The main point of interest in this behavior is the fiber stresses resulting from differential shrinkage, that is, the differential effect of shrinkage of the cast-in-place concrete and shrinkage-plus-creep of the prestressed concrete, while the two are bonded together. The strain-distribution values in Figure 8c, when multiplied by the modulus of elasticity of the stem concrete, lead at once to the differential-shrinkage stress distribution in the stem of the composite section.

But this is not the case in the slab, where the actual slab strain is a net shortening from shrinkage, and the shrinkage strains themselves are not accompanied by concrete stress. It is the resistance of the stem to the slab shrinkage which produces slab stresses. The strain corresponding to this stress, which is the same as the decrease in free slab shrinkage strain resulting from the stem restraint, is shown in Figure 8d.

Reinforcing steel also provides a restraint on slab shortening and produces tensile stresses in the concrete, but this can be considered separately provided the coefficient defining the magnitude of the differential-shrinkage effect has been properly determined for such reinforcement. It is the resistance of the stem to slab shortening which causes the differential shrinkage stresses to be introduced in the slab. Hence the effect of differential-shrinkage is to exert a compressive force at the top of the stem and an equal tensile force at the bottom of the slab. The stresses imparted to each portion of the composite beam by these equal forces are easily calculated, once the force is known.

An important deduction must be made regarding the strain distribution in both portions of the composite section. As the differential-shrinkage occurs, the entire beam bends downward. Assuming perfect bond between the two sections, the curvature of the composite section is given by

$$\text{Curvature} = 1/R = \epsilon/y.$$

Hence is follows that the strain distribution is linear across the composite section. But this compressive strain in the slab is the actual slab strain from shrinkage. It does not reflect the decrease in the free slab strain caused by the stem restraint, which

corresponds to the differential shrinkage stresses in the slab. Therefore, the stress distribution in each section and its corresponding strain distribution (Fig. 8d) do not necessarily have the same slope but depend rather on the individual properties of the respective cross sections. An example of this is given in the summary.

A question might be asked here about the need for a transformed composite section to be used in the determination of the different-shrinkage stresses in order to allow for different moduli of elasticity in the two sections. There is no external bending moment on the composite section as a result of differential-shrinkage and hence no internal resisting couple. The differential-shrinkage force acting in one direction on the top of the stem and in the opposite direction on the bottom of the slab exert bending moments on their respective sections of Pd/2. These moments are different when the two section depths are different. Each moment is resisted by its respective section and not by a combined transformed composite section, since no moment acts on the composite section itself as a result of differential-shrinkage. This suggests that the stress and strain distribution in each portion of the composite section depends on the properties of that section alone, and the curves do not necessarily have the same slope.

The foregoing discussion and the equations to be developed for the calculation of the differential shrinkage force are for a composite T-beam, but they also apply to any composite prestressed-concrete beam regardless of the cross section used.

In presenting the theory for the calculation of differential-shrinkage stresses, it is desirable to use a variable coefficient which is independent of the two cross sections used in the composite beam. For this reason the differential-shrinkage coefficient "x", as defined, is used instead of a simpler value such as one of the extreme fiber strains. The differential-shrinkage coefficient is defined as the difference in the free strains of the two sections It is also the sum of the actual strain of the bottom of the slab (this is equal to the actual top-fiber strain of the stem) and the tensile strain of the bottom of the slab caused by the stem restraint. This is the same as the relative strain of the bottom of the slab and the top of the stem if the two sections were not bonded together; that is, the difference in the free strains.

In this investigation the value of "x" was obtained from the experimentally determined strain distribution of the beam portion of the composite section. Using this value for the coefficient, the differential-shrinkage force and the corresponding extreme fiber stresses of both sections were calculated. It is seen that this approach offers an easy way to calculate differential shrinkage stresses provided an appropriate coefficient is known. For design purposes it seems feasible to use such a coefficient depending on the difference between the casting dates of the two concretes. More experimental work is needed to determine satisfactory values for the coefficient under various casting schedules.

All stresses and strains are those resulting from differential shrinkage alone (the differential effect of shrinkage of the cast-in-place concrete and shrinkage-plus-creep of the prestressed concrete, while the two are bonded together).

The sign convention is based on the assumption that the slab shrinkage exceeds the shrinkage-plus-creep of the top fiber of the beam. Otherwise the signs are reversed.

Subscripts 1 and 2 refer to the cast-in-place and prestressed sections respectively. Subscripts t and b refer to top and bottom fibers respectively.

After the two sections are bonded together, the differential shrinkage induces a compressive force P_2 on the top fiber of the beam. This is opposed by a tensile force P_1 on the bottom fiber of the slab section.

$\Sigma M = 0$, P_1 coincides with P_2

$\Sigma H = 0$, $P_1 = P_2 = P =$ Differential Shrinkage Force

x = Differential-Shrinkage Coefficient (experimentally determined)

= (free-shrinkage strain of the bottom fiber of the slab)—(Free-shrinkage-plus-creep strain of the top fiber of the beam)

= $(\epsilon_{1b}) + (\epsilon_{2t})$ = (Tensile strain in the bottom fiber of the slab caused by the restraining action of the beam on the relative slab shortening, that is, differential-shrinkage) + (Compressive strain in the top fiber of the beam because of the same restraining action)

$$a_1 = \frac{E_1}{1/A_1 + (\frac{1}{2}d_1)2/I_1},$$

$$a_2 = \frac{E_2}{1/A_2 + (\frac{1}{2}d_2)^2/I_2}$$

The a-values represent the resistance properties of the two sections as defined below.

$$\sigma = \epsilon E$$
$$\sigma = P/A + Pec/I$$

$$P = \frac{}{1/A + (\frac{1}{2}d)^2/I}$$

$$= \frac{\epsilon E}{1/A + (\frac{1}{2}d)^2/I} = \epsilon^a$$

$$P_1 = \epsilon_{1b}a_1, \qquad P_2 = \epsilon_{2t}a_2$$
$$P_1 = P_2 = P$$
$$\epsilon_{1b}/\epsilon_{2t} = a_2/a_1$$
$$x = \epsilon_{1b} + \epsilon_{2t} = \epsilon_{2t}(\epsilon_{1b}/\epsilon_{t2} + 1)$$
$$= \epsilon_{2t}(a_2/a_1 + 1)$$

$$\epsilon_{2t} = \frac{xa_1}{a_1 + a_2}$$

$$P = \epsilon_{2t}a_2 = \frac{a_1 a_2}{a_1 + a_2} x$$

The differential-shrinkage force is given by this formula. While the differential-shrinkage coefficient x is independent of the sections used, the differential shrinkage force P depends on the coefficient, the two moduli of elasticity, and the two cross sections.

$$\sigma = P(\pm 1/bd \pm \frac{(d/2)^2}{bd^3/12}) = P/bd \pm 1 \pm 3)$$

$$\sigma_{1t} = -2P/b_1 d_1,$$
$$\sigma_{2t} = -4P/b_2 d_2$$
$$\sigma_{1b} = 4P/b_1 d_1,$$
$$\sigma_{2b} = 2P/b_2 d_2$$

Calculation of Differential Shrinkage Stresses

$d_1 = 4$ in.,
$A_1 = 144$ in^2.,
$I_1 = 192$ in^4.,
$E_1 = 2.9 \times 10^6$ psi
$d_2 = 14$ in.,
$A_2 = 140$ in^2.,
$I_2 = 2287$ in^4.,
$E_2 = 4.1 \times 10^6$ psi

From Figure 6, the ultimate value of ϵ_{2t} was estimated to be about 118 x 10^{-6} in./in.

$$a_1 = \frac{2.9 \times 10^6}{1/144 + 4/192} = 1.044 \times 10^8 \text{ lb.},$$

$$a_2 = \frac{4.1 \times 10^6}{1/140 + 49/2287} = 1.435 \times 10^8 \text{ lb.}$$

$$x = 118 \times 10^{-6}(1.435/1.044 + 1)$$
$$= 2.802 \times 10^{-4} \text{ in./in.}$$

$$P = \frac{1.044 \times 1.435 \times 10^{16}}{(1.044 + 1.435)\,10^8}$$
$$2.802 \times 10^{-4} = 16{,}930 \text{ lb.}$$

This is the differential shrinkage force.

$$\sigma_{1t} = -2 \times 16{,}930/36 \times 4 = -235 \text{ psi C.}$$
$$\sigma_{1b} = +470 \text{ psi T.}$$
$$\sigma_{2t} = -4 \times 16{,}930/10 \times 14 = -484 \text{ psi C.}$$

The product of the top-fiber strain of the prestressed section and the modulus of elasticity provide a check on the calculations.

$$\sigma_{2t} = \epsilon_{2t}E_2 = -118 \times 10^{-6} \times 4.1 \times 10^6$$
$$= -484 \text{ psi C.}$$
$$\sigma_{2b} = +242 \text{ psi T.}$$

By using the calibration between the stem gage readings and the corresponding longitudinal force applied at the top of the stem, the average differential-shrinkage force, as indicated by the estimated ultimate differential shrinkage strain distribution based on the strain measurements (Fig. 8) at the four stem gages, was found to be 17,600 lb. This compares favorably with the magnitude of differential-shrinkage coefficient.

Calculation of Ordinary Slab-Shrinkage Stresses

Reinforcing was symmetrically placed at mid-depth.

$$f_t = \frac{np}{1 + (n\text{-}1)p} sE_c$$

For the slab used,

$n = 10$, $p = 7 \times 0.20/144 = 0.00973$,
$E_c = 2.9 \times 10^6$ psi,
$s = 400 \times 10^{-6}$ in./in.
$f_t = 104$ psi.

SUMMARY

The magnitude of the estimated ultimate shrinkage strain was found from the experimental data to be about 4 x 10^{-4} in./in. This is higher than the usual recommended value for field designs but is approximately the value to be expected under laboratory conditions. This results in an ultimate loss of prestress from shrinkage of 9.5 percent. The creep ratio found in the noncomposite beam was 1.1 which is a normal value, but the ultimate loss of prestress of 14.7 percent from creep is high. The explanation for this seems to lie in the relatively high elastic-concrete strain (13.8 percent of the initial steel strain), or high prestress force used. This resulted in a lower creep ratio since for stress ranges above about 1800 psi, creep is no longer proportional to stress. However, if this one test is any indication, it appears that a 15 to 20 percent loss of prestress from concrete shrinkage and creep alone might be found in the field. This is higher than is normally recommended for design purposes.

From the noncomposite prestressed-concrete beam, it was found that creep was proportional to stress up to about 0.3 of the ultimate strength of the concrete.

The strain distribution resulting from differential-shrinkage was obtained from experimental data, and an easy method shown for calculating the differentian-shrinkage force and the corresponding stresses in both portions of the composite section. The method is one of determining the differential-shrinkage force for a given differential shrinkage coefficient, using the formula

$$P = \frac{a_1 a_2}{a_1 + a_2} x,$$

and applying the ordinary (P/A + Pec/I) formula to calculate the fiber stresses. Such a solution insures linear stress and strain distributions in each portion of the composite section as a result of differential-shrinkage but not necessarily the same slope in each. This behavior differs from the assumption made in the first two references in the bibliography that the strain-distribution diagram corresponding to the differential-shrinkage stresses is linear across the composite section. The consequence is a difference in the top-fiber stress of the slab and the bottom-fiber stress in the stem.

Referring to the differential-shrinkage theory, it is seen that for the two portions of the composite section, the numerical sum of the extreme fiber stresses is $6P/b_1d_1$ for the slab and $6P/b_2d_2$ for the stem. It follows from these that, unless $b_1d_1^2 = b_2d_2^2$, the stress distribution have different slopes in the two sections. If this equality happens to hold in any given case and Hook's Law is assumed, then the strain-distribution diagrams would have the same slope only if the two moduli of elasticity were the same. This supports the statement previously made that the stress and strain distribution in both portions of the composite section do not necessarily have the same slope as a result of differential-shrinkage. The strain referred to here is not the actual slab strain, which includes the shortening resulting from shrinkage, but rather the strain corresponding to the slab stress caused by the resistance of the stem to slab shortening.

In these tests one value for the coefficient was found to be 3.1 x 10^{-4} in./in., and the corresponding differential-shrinkage force and fiber stresses were calculated. They were of high enough magnitude to require consideration in design. The resulting force was substantiated experimentally by a calibration made between the stem gages and an applied longitudinal force at the top of the stem. The differential-shrinkage effect is high insofar as the difference between the casting dates (104 days) of the two concretes is concerned. Also the conditions of field shrinkage would call for a lower coefficient, but the reduction would be less than that for the case of free shrinkage in the laboratory and the field. More test beams are needed to determine the coefficient for various casting schedules.

The main variable effecting differential shrinkage in most cases is no doubt the elapsed time between the casting dates. Four other variables were mentioned which are significant in the evaluation of differential-shrinkage. They were (1) the surrounding moisture conditions, (2) whether the slab is reinforced or not, (3) the two concrete mix designs, and (4) the amount of prestress used. The prevailing moisture conditions in the area of a structure effect the differential-shrinkage coefficient much as they do the free shrinkage of plain con-

crete. Extreme conditions can be provided for in the coefficient itself. However, curing conditions no doubt have a considerable effect on the coefficient which should be used in a design. Further investigation of this is needed. The magnitude of the results found herein were based on data resulting from tests on elements which were cured for one week. This time interval might be considered as a normal construction procedure, hence this variable could be eliminated from among the previously mentioned list.

It may be assumed that the slab is reinforced for most cases as they generally are. The mix designs for the two concretes are fairly well standardized (especially in bridge designs), so that this would reduce the variables further. The normal distribution of prestress is not likely to vary enough to alter the creep distribution materially. Hence it appears that a reasonably simplified analysis could be achieved in which the differential shrinkage coefficient is based on the estimated time between casting of the two concretes. The longer this time interval, the greater are the differential-shrinkage stresses in both sections. For this investigation the tensile stress in the slab caused by ordinary shrinkage, plus differential-shrinkage, was calculated to be $104 + 470 = 574$ psi at the bottom fiber. There was no evidence of the slab cracking after 100 days.

CONCLUSIONS

The free concrete shrinkage data obtained under laboratory conditions indicate a slower rate of concrete shrinkage than is usually reported.

The strain distribution diagrams corresponding to the differential-shrinkage stresses do not necessarily have the same slope in each portion of the composite beam. The differential-shrinkage force produces moments about the centroid of each section in the composite beam, and this combined moment and longitudinal force induce stresses in each section as determined by the properties of the individual section itself, and not by a transformed composite section.

The stress distribution resulting from differential-shrinkage and slab dead load cause a creep distribution different from that of the same beam without the cast-in-place slab. However, at the center of gravity of the prestessed steel this change in stress is very small so that the change in creep at that point — and thus the change in prestess loss because of the change in concrete creep — is negligible in the composite beam. Hence the loss of prestress from concrete creep is not altered from that of a non-composite beam by the slab dead load or differential-shrinkage effect.

The procedure developed herein provides a simple method for calculating the stresses resulting from differential-shrinkage, if an appropriate differential-shrinkage coefficient is known. For design purposes it is believed that this coefficient can be prescribed for a given estimated period of time between casting of the two concretes. Additional information is needed on this and other aspects of composite differential shrinkage.

The only difference in the differential shrinkage coefficient for the case of pretensioned beams is that the coefficient should be slightly larger because of the larger percentage of creep in the prestressed portion which occurs before the slab is cast. This results in a larger differential effect and depends on the time of release of prestress for pretensioned beams and the time of applying the post-tensioning force But for most designs the difference in the coefficient to be used for pre- and post-tensioned beams is negligible.

ACKNOWLEDGMENT

These tests were conducted as part of a research program sponsored by the Florida State Road Department. The post-tensioning bars and the flexible tubing were donated by Stressteel Corporation and Flexico Products, respectively. The authors wish to thank Messrs. Dick Tadjer, Joe Diniz and Al Conyers for their assistance.

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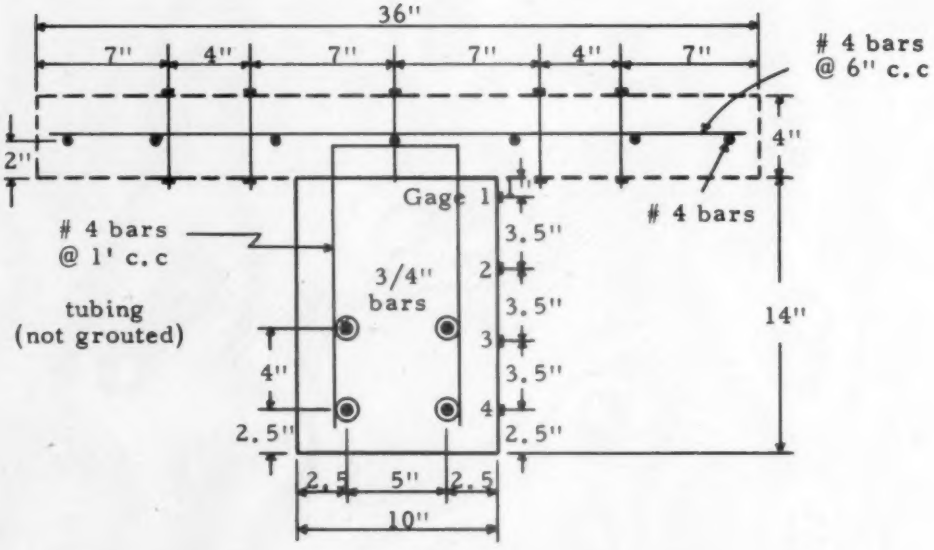
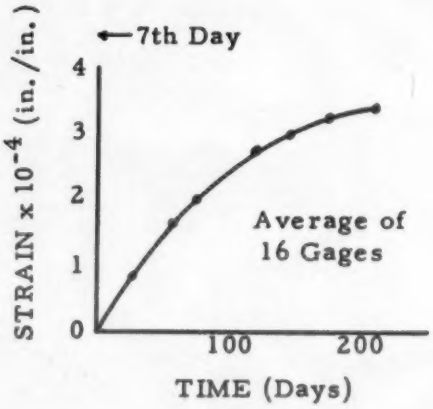
Fig. 1 Composite and Non-Composite prestressed concrete beams

Fig. 2 Whittemore Mechanical Strain Gage

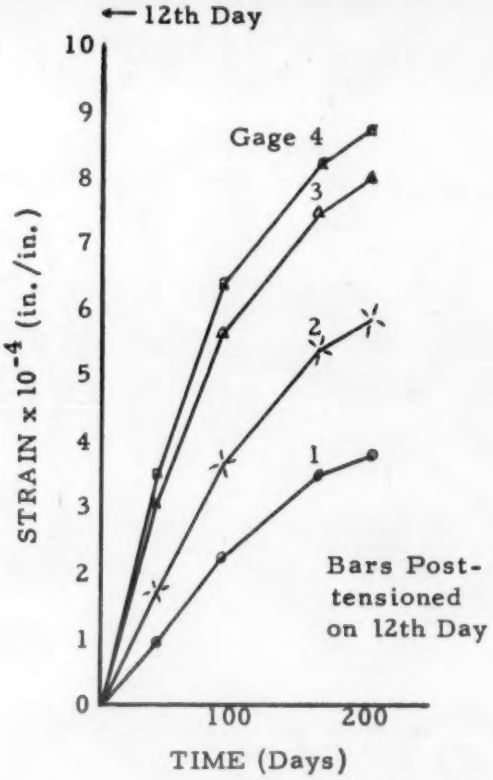


Location of the stem gages are the same for the non composite beam and the shrinkage specimens

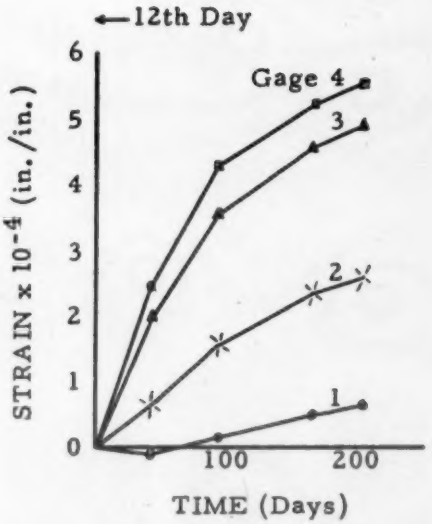
Figure 3. Composite Beam



a. Shrinkage Strain of Shrinkage Specimens

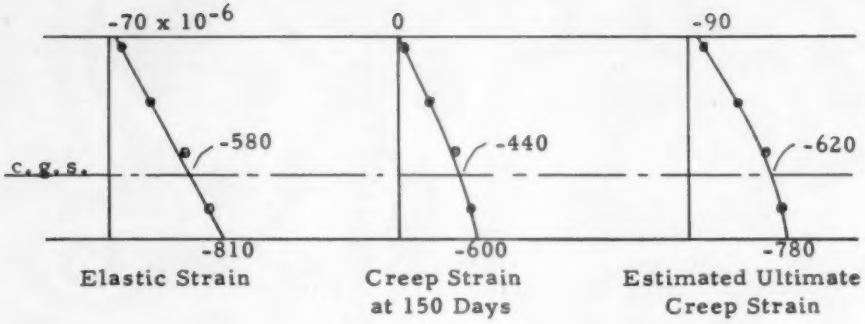


b. Combined Shrinkage and Creep of Non Composite Beam



c. Net Creep of Non Composite Beam

Figure 4. Shrinkage and Creep Strains



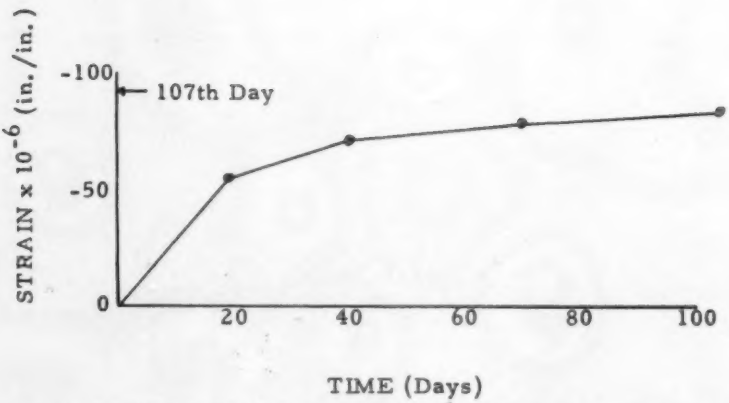


Figure 6. Average Bottom Fiber Slab Strain Minus the Top Fiber Strain of the Non Composite Beam (From the 3rd Day After Slab Casting.)

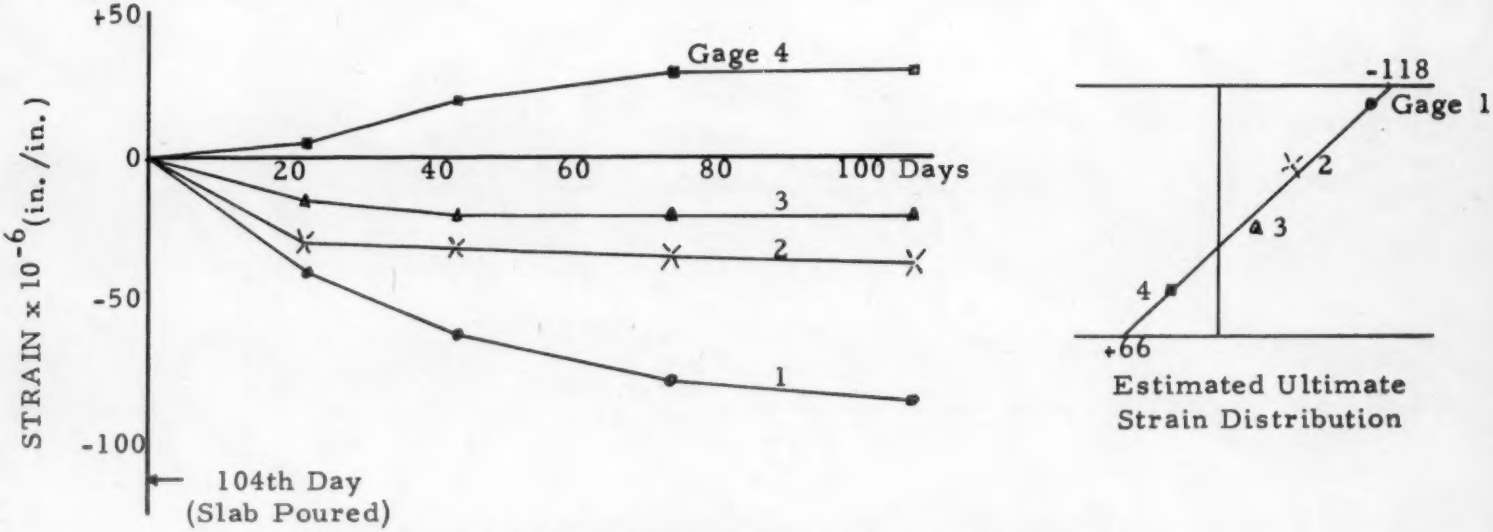


Fig. 7 Differential shrinkage strains (Composite Beam strains minus the Non-Composite Beam strains (From the time of slab casting))

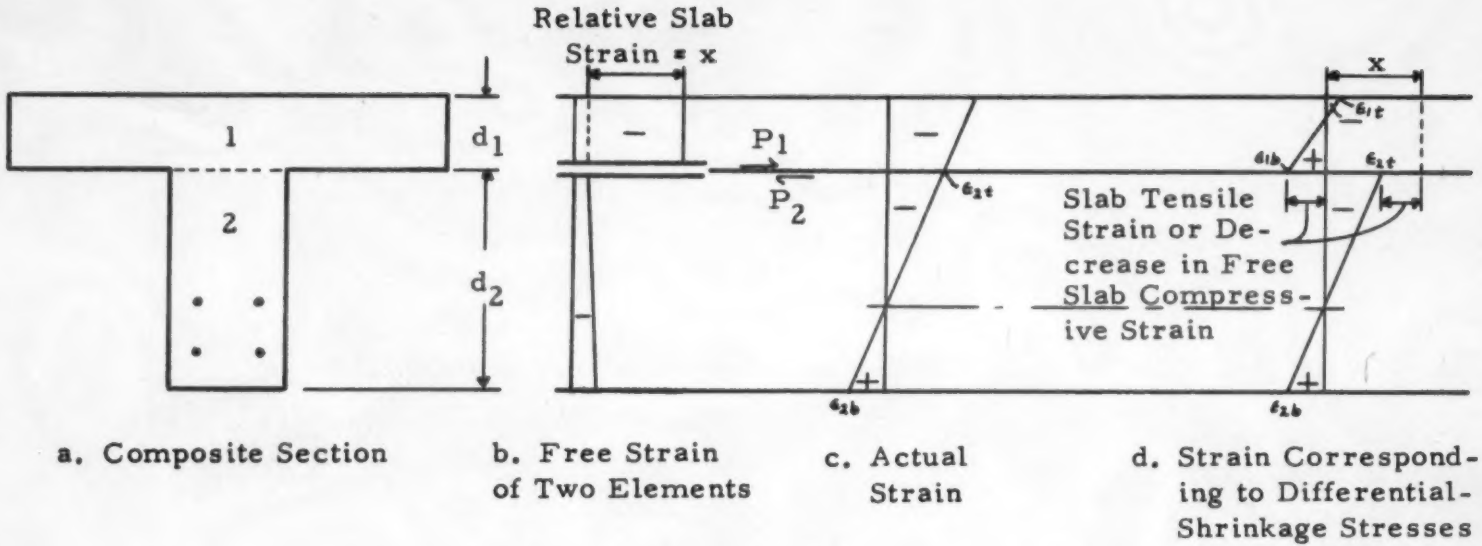


Fig. 8 Composite Section and strain distribution resulting from concrete shrinkage and creep
(From the time of slab casting.)

APPENDIX

TEST SETUP

Two concrete speciments, 3½ in. by 5 in. by 6 ft., were used to investigate the expansion and shrinkage behavior of concrete beginning immediately after casting. A steel bar was embedded in each specimen to measure the strains resulting from the heat of hydration and concrete shrinkage. One bar was unsymmetrically placed in an effort to separate the expansion and contraction effects and the shrinkage effects which occur simultaneously during the first few days after casting.

Before the concrete was poured, the bars were anchored to rigid abutments, and an initial tensile strain of about 100 x 10^{-6} in./in. was introduced in each by tightening the nuts on the ends of the bars. The initial tension was to prevent slippage and to allow for the effect of heat of hydration produced after the concrete was bonded to the steel bars. This heat caused the embedded steel to elongate and the external steel to shorten thus releasing some of the initial tension in the external part of the bars. After the concrete shrinkage and cooling effect exceeded the expansion the reverse was true. In either case the strains for the concrete and embedded steel were opposite in sign to those for the external part of the steel bars where the gages were located.

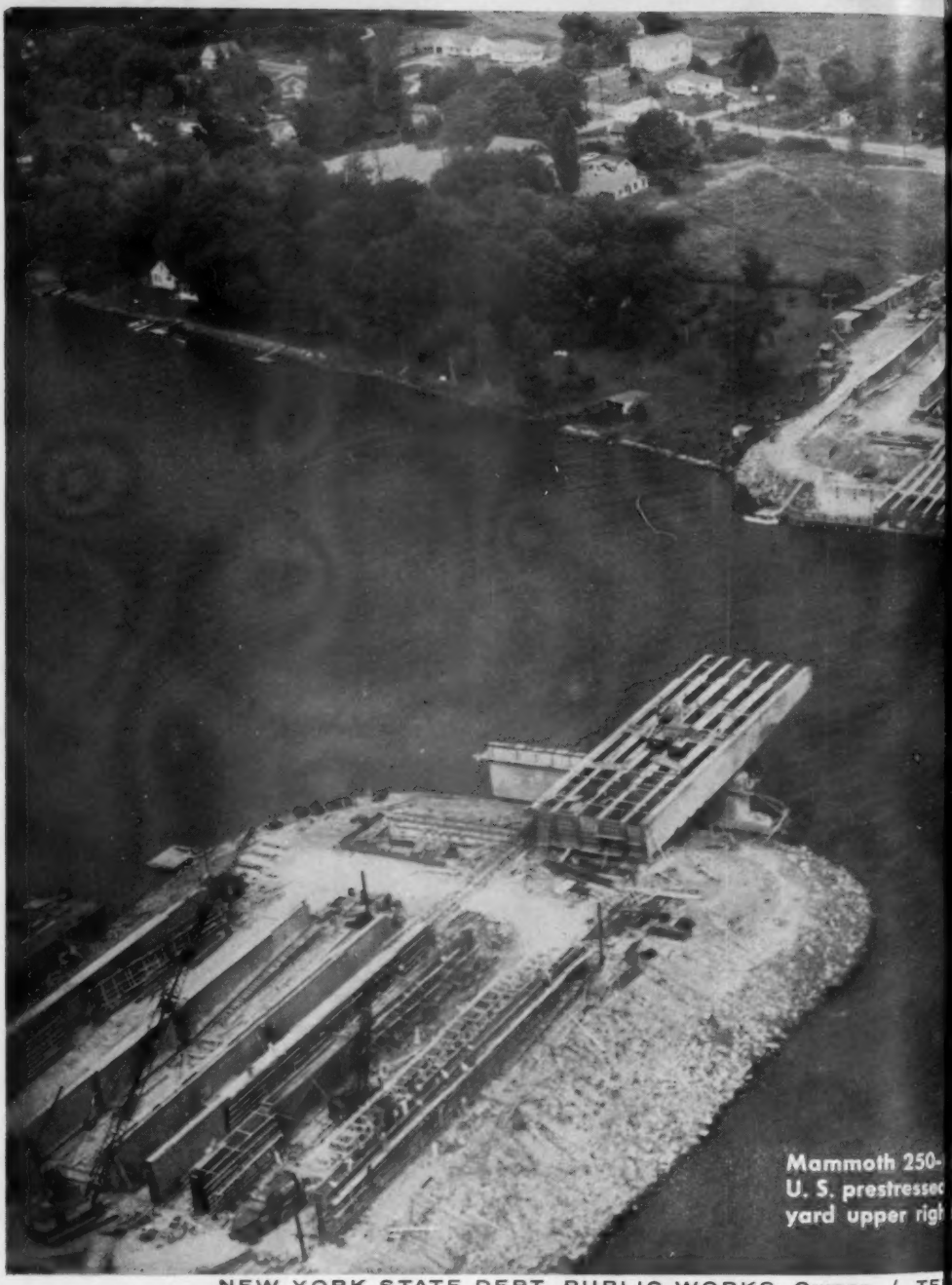
Electrical strain gages were placed near each end of the steel bars outside the concrete forms. Expansion material was used inside the wood forms so that expansion and shrinkage of the concrete would be unimpeded. The strain readings were taken as often as needed beginning immediately after casting and continuing for 20 days. The specimens were cured for 2 days and the forms removed after 5 days. Average values for the strain readings on both ends of each bar are reported.

ANALYSIS OF TEST RESULTS

The curves in Figure 11 show the readings of the electrical strain gages for each specimen. After the casting of the concrete, there was a 2½ hour period in which no change was noted in the initial gage readings. Before the concrete was bonded to the steel, any heat caused by hydration of the concrete would tend to relax the initial tensile stress in the bar, but since the bar could not change in length, there was no measurable strain. At 2½ hours an abrupt change occurred in the gage readings indicating the concrete had begun to bond rather rapidly to the steel. Evidently the initial set of the concrete occurred about this time, and the resulting heat of hydration began to elongate the concrete and embedded steel. Since the coefficient of thermal expansion of concrete is about 0.9 that of steel, it appeared that any differential effect from the expansion and contraction of the concrete and steel as a result of heat hydration was negligible.

After 2½ hours the embedded steel strains and concrete strains were the result of a combination of expansion caused by the heat of hydration followed by contraction upon cooling and contraction resulting from concrete shrinkage, which began about the same time that the concrete started bonding to the steel. From 2½ to 7 hours after casting it was found that the effect of expansion of the embedded steel and concrete caused by heat was predominate over that of the concrete shrinkage. After 7 hours, concrete shrinkage and cooling caused the members to shorten. The maximum unit elongation of the concrete caused by heat of hydration was not excessive (29 x 10^{-6} in./in.) and, since its effect is cancelled on cooling, no account need be taken of it. In both pre-tensioned and post-tensioned members, the expansion and contraction effects have normally cancelled themselves before the prestress is applied to the concrete.

From 7 to 24 hours the specimen with the symmetrically placed steel shortened and returned to a net zero strain. Evidently after 7 hours the heat of hydration had caused its maximum expansion, and cooling plus concrete shrinkage began to shorten the member. This time interval would no doubt vary considerably with the size on member and the concrete mix used. The relative part that concrete shrinkage plays in this shortening of the member can be perceived intuitively by comparing the two curves in Figure 11a. The shrinkage of the concrete did not exert as great a force on the unsymmetrically placed steel as it did on the symmetrical specimen, so that the symmetrical specimen returned to a net zero strain after one day and the



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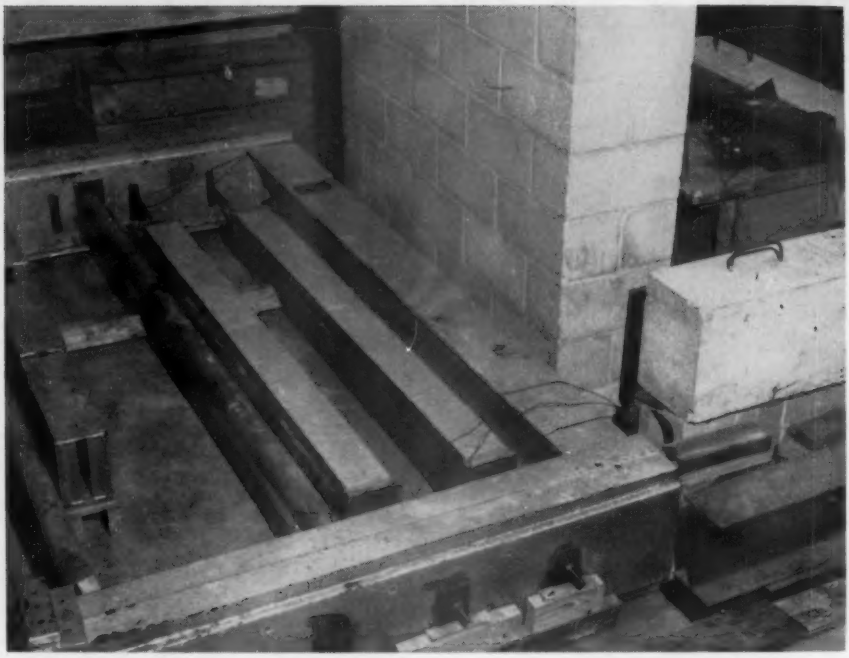


Fig. 9 The two specimens and rigid abutments (Also shown is the strain indicator and switching unit at right)

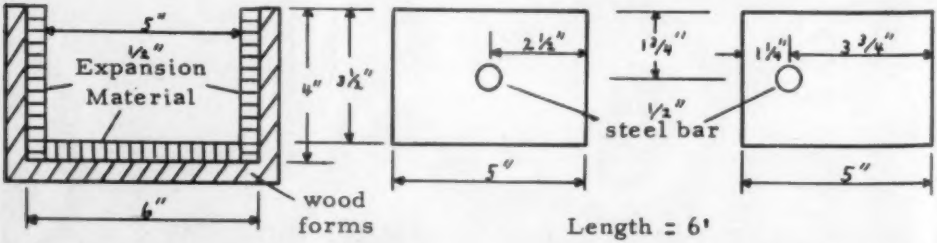
unsymmetrical specimen after two days.

The exact time when the cooling was completed is not important because the concrete and steel strains caused by heating and cooling cancel each other after a few days, leaving only the concrete shrinkage strains (Figure12). It may be said, however, that the predominate effect during the first 24 hours was heating and cooling from the hydration of the concrete, although concrete shrinkage was taking place during this time.

It is the concrete shrinkage that produces stresses in the concrete and steel in the case of reinforced concrete members and differential shrinkage stresses in composite members, not the expansion and contraction caused by heating and cooling. The concrete shrinkage begins right after the initial concrete set and bonding of the concrete to the steel.

The free shrinkage curve in Figure 12, which was measured from the time of casting, is to the left of the curve in Figure 4, indicating an increase in the ultimate shrinkage strain of about 60 x 10^{-6} in./in.

Fig. 10 Two specimens and forms.



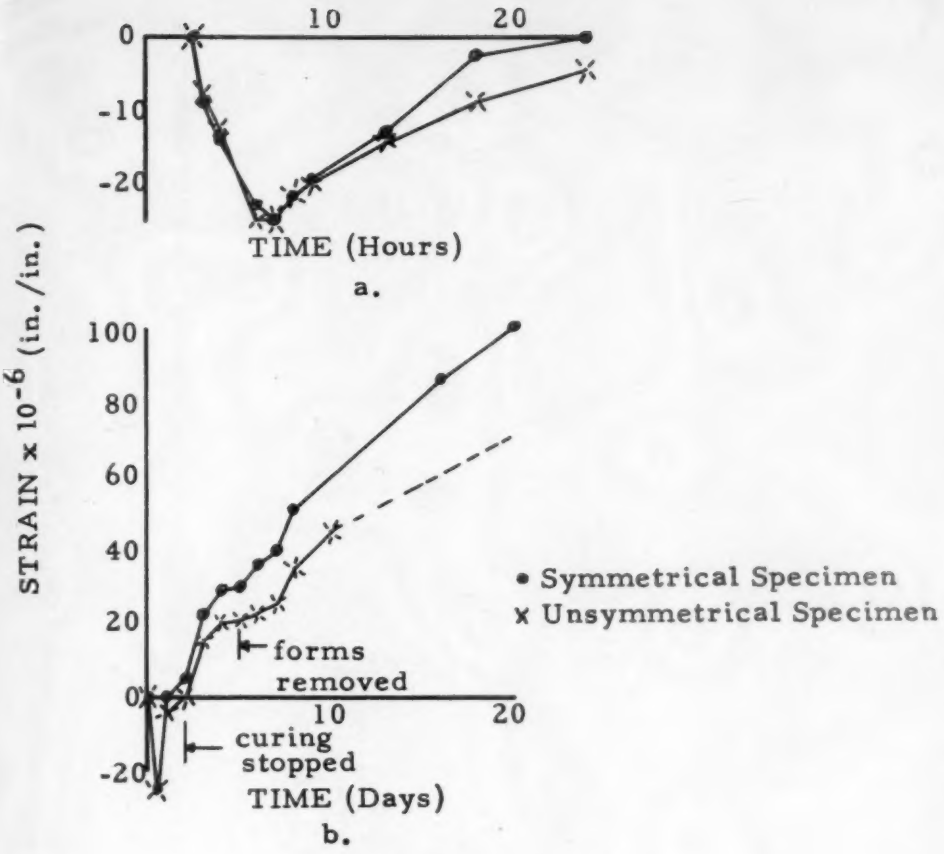


Fig. 11 Strain gage readings

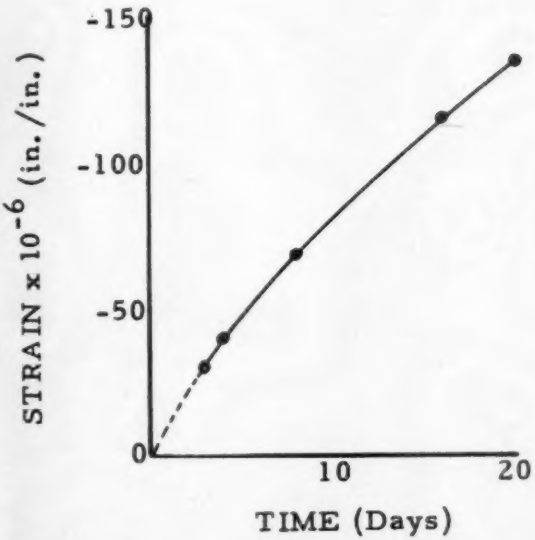


Fig. 12 Free concrete shrinkage strains

A Deflection Theory For Rectangular Prestressed Concrete Beams After Flexure Cracking Has Commenced . . .

By

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SYNOPSIS

A theory for deflection of prestressed concrete beams is proposed which applies from the onset of cracking up to the load that produces non-linear stress distribution in either the concrete or the steel. Prior to cracking, Navier's fundamental differential equation for deflection is applicable.

Experimental tests were carried out on twenty-six 6″ x 4″ x 10′0″ pretensioned beams subjected to a large range of initial concrete prestress, from 30% to 300% of that normally used in practice. The beams were tested to failure at age of about 36 days by the continuous application of incremental loading. Good correlation between the observed deflections and those calculated from the proposed theory were obtained.

INTRODUCTION

The fundamental differential equation for beam deflection due to flexure was proposed by Navier[1] in 1826. This equation applies to beams of most materials but when applied to concrete, differences of opinion occur as to the value of the modulus of elasticity of concrete (E_c) and the moment of inertia of the beam cross-section (I) or the combined flexural rigidity term E_cI. In this investigation most factors affecting E_c were restricted and for a particular strength concrete, the modulus of elasticity was found to be reasonably constant. Prior to cracking, the whole beam cross-section is operative and Navier's differential equation, $\frac{d^2y}{dx^2} = \frac{M}{E_cI}$ is valid provided that the moment of inertia of the whole area (I_D) is used.

Discussion on the accurate determination of deflection prior to cracking is not dealt with at length in this paper.

When cracking commences the neutral axis position shifts and in the case of the non-prestressed beam rises suddenly to a constant position, predicted by normal reinforced concrete theory. In the pretensioned beams however, the prestressing force opposes the commencement of cracking and retards the rate of crack development as the bending moment increases. The effective cross-section therefore is a function of the prestress force and the applied bending moment.

As an alternative to using the moment of inertia of the cracked cross-section to obtain the deflection, the beam curvature $\frac{M}{EI}$ can be expressed in terms of strains or stresses as suggested by Maney in 1914[2] and 1926[3]. The proposed theory for deflection of prestressed beams in the cracked range is divided into two parts.

(1) The determination of the expressions for the extreme fibre compressive stress and the steel stress in terms of the cracked section properties and the applied loads.
(2) The integration of these expressions for stress along a beam to obtain the deflection.

NOTATION

The notation proposed by the A.C.I.-A.S.C.E. Committee 323[4] is used whenever possible but as some alterations are necessary the following list is included. Subscripts 1 and 2 refer to the top and bottom (fibres) respectively.

Cross-sectional properties.

b = width of the rectangular section.
d = distance from the extreme compressive fibre of the line of action of the prestressing force.
kd = distance from the extreme compressive fibre to the beam neutral surface.
qd = depth of beam that is uncracked.
D = overall depth of beam.
h = distance from the extreme compressive fibre to the layer that is unstressed by the externally applied bending moment.
A_s = area of prestressing steel.
A_c = total prestressed area of concrete prior to cracking and equals $bD + (n-1)A_s$ for a rectangular beam.
I = moment of inertia of the beam cross-section in general.
I_D = moment of inertia of the total beam area A_c about the centroid of the cross-section.
Z_1 = section modulus of the uncracked cross-section related to the top fibre.
Z_2 = section modulus of the uncracked cross-section related to the bottom fibre.
P = effective prestressing force.
e = eccentricity of P from the centroid of the area A_c.

Beam Loading Notation.

M = externally applied bending moment in general.
M_c = theoretical cracking bending moment and is the bending moment at which the concrete tensile stress exceeds f'mr at any cross-section of a beam.
M_L = limit of stress linearity bending moment and is the bending moment at which the beam stress distribution of either the concrete or steel becomes non-linear.
M_u = ultimate bending moment and is the bending moment at which a beam collapses.

Concrete properties.

f'_c = ultimate compressive stress of concrete at time of beam test on a 6″ dia. x 12″ high cylinder.
f'_{mr} = modulus of rupture tensile stress obtained from an unreinforced 6″ x 4″ x 9′0″ beam similar to those tested.
f_{P1} = extreme top fibre stress due to prestress force.

f_{P2} = extreme bottom fibre stress due to prestress force.
f_{L1} = extreme compressive fibre stress due to external loading.
f_{c1} = extreme compressive fibre stress due to all actions and equals ($f_P{}^1 + f_{L1}$).
e_{c1} = extreme compressive fibre strain due to f_{c1}.
E_c = flexural modulus of elasticity of concrete.

Steel properties.

f'_s = ultimate stress of prestressing steel.
f_{se} = effective steel prestress.
f_{sa} = additional steel stress due to externally applied bending moment M.
f_s = steel stress due to all actions and equals ($f_{se} + f_{sa}$).
e_s = steel strain due to f_s.
E_s = modulus of elasticity of the prestressing steel.
n = ratio of moduli of elasticity of steel to concrete and equals $\frac{E_s}{E_c}$.

GENERAL THEORY

In developing the theory the following assumptions are made.

(1) Concrete is assumed to be capable of sustaining tensile stresses up to a value equal to f'_{mr}.

(2) Beyond this limiting tensile stress the effect of the tensile concrete is neglected and the beam is assumed to have a corresponding effective transformed area.

(3) The strain distribution across the beam remains linear throughout the loading range.

(4) The materials of the beam remain within their elastic limits. While the beam strain distribution remains linear the differential equation for deflection can be expressed in terms of strain.

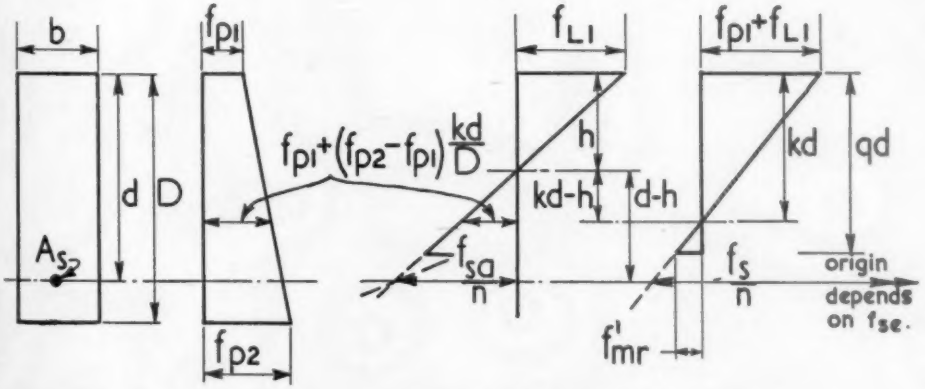


Fig. 1

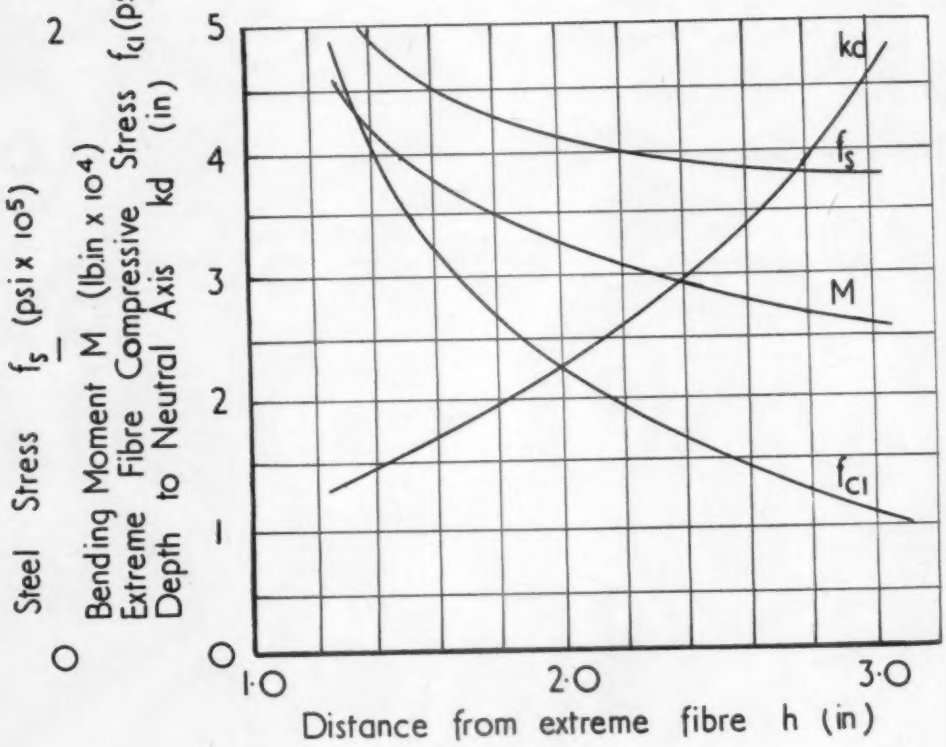

Steel Stress f_s (psi x 10^5)
Bending Moment M (lb.in x 10^4)
Extreme Fibre Compressive Stress f_{c1} (psi x 10^3)
Depth to Neutral Axis kd (in)
0
1
2
3
4
5
1·0
2·0
3·0
kd
f_s
M
f_{c1}
Distance from extreme fibre h (in)

Fig. 2

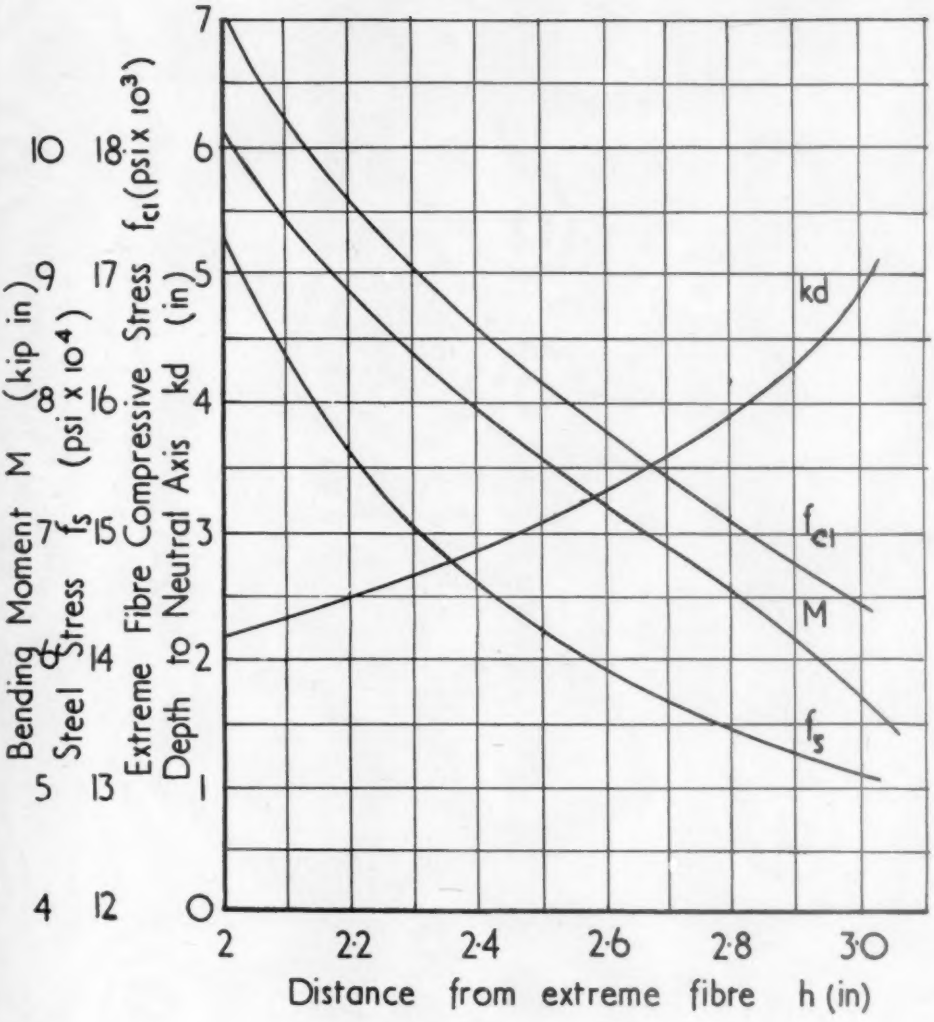

Bending Moment M (kip in)
Steel Stress f_s (psi x 10^4)
Extreme Fibre Compressive Stress f_{c1} (psi x 10^3)
Depth to Neutral Axis kd (in)
10
9
8
7
6
5
4
18
17
16
15
14
13
12
7
6
5
4
3
2
1
0
kd
f_{c1}
M
f_s
2
2·2
2·4
2·6
2·8
3·0
Distance from extreme fibre h (in)

Fig. 3

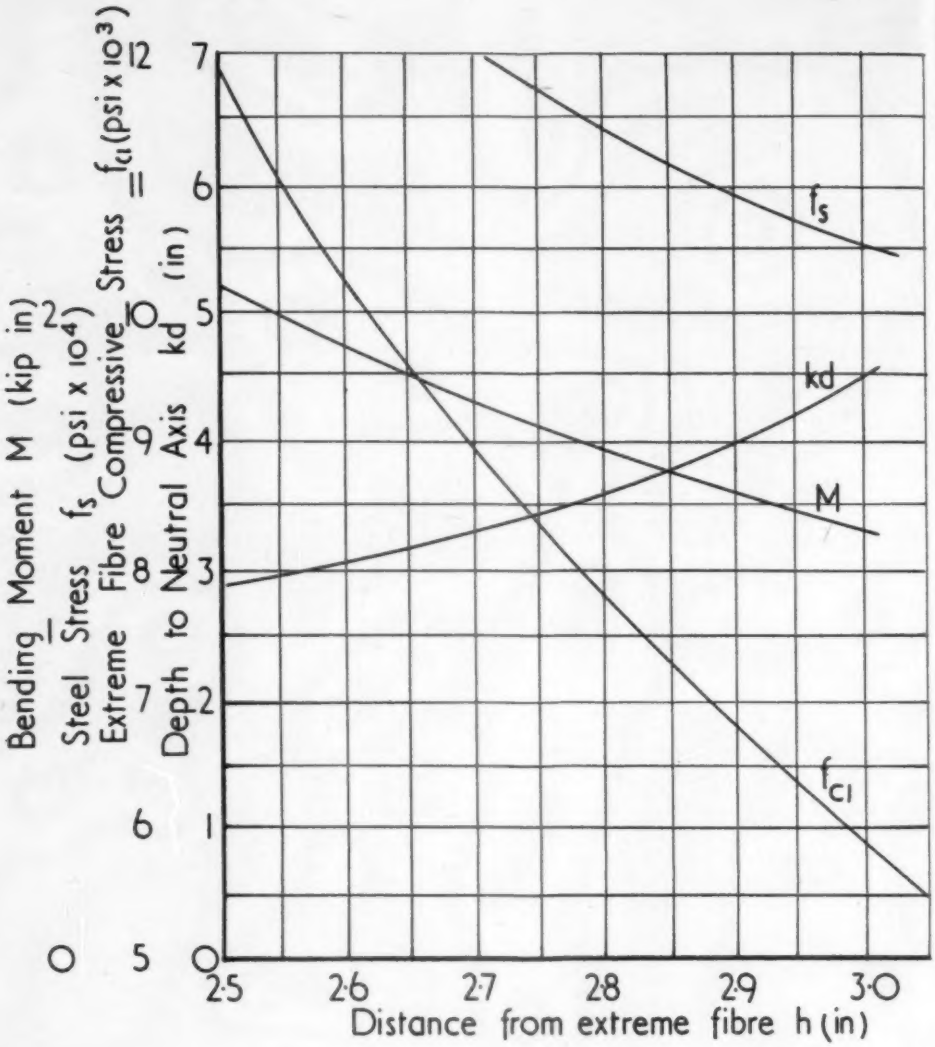


Fig. 4

$$\frac{d\Theta}{dx} = \frac{(e_s + e_{c1})}{d} \quad \ldots\ldots (1)$$

and assuming that the materials remain within their elastic limit the equation in terms of stress becomes,

$$\frac{d\Theta}{dx} = \frac{d^2y}{dx^2} = \frac{1}{dE_s}(f_s + nf_{c1}) \quad \ldots\ldots (2)$$

The usual $\frac{M}{E_cI}$ diagram can then be replaced by a $\frac{1}{dE_s}(f_s + nf_{c1})$ diagram and twice inte-

grated to determine the deflection at any point along a beam. This operation is relatively simple, the main difficulty being the calculation of the stresses as the cracking develops.

The extreme fibre stresses due to an effective prestress force P acting at an eccentricity e on a rectangular cross-section are obtained from Eq. (3) and (4).

$$f_{P1} = \frac{P}{A_c} - \frac{Pe}{Z_1} \qquad \ldots\ldots (3)$$

$$f_{P2} = \frac{P}{A_c} + \frac{Pe}{Z_2} \qquad \ldots\ldots (4)$$

Stress determination after cracking commences.

Let an external bending moment be applied to a beam such that the extreme tensile fibre stress exceeds the modulus of rupture value f'_{mr}. This will cause the concrete to crack to a depth distant qd from the extreme compression fibre as shown in Fig. 1(d) where the concrete tensile stress equals f'_{mr}.

From the geometry of Fig. 1(c)

$$f_{L1} = \frac{h}{kd\text{-}h}\left[f_{P1} + (f_{P2} - f_{P1})\frac{kd}{D}\right] \qquad \ldots\ldots (5)$$

From the geometry of Fig. 1(d).

$$qd = \frac{f'_{mr}\,kd}{f_{L1} + f_{P1}} + kd \qquad \ldots\ldots (6)$$

The additional concrete stress at the steel line due to external bending moment is obtained from Fig. 1(c).

$$f_{sa} = nf_{L1}\left(\frac{d\text{-}h}{h}\right) \qquad \ldots\ldots (7)$$

The total steel stress at any stage of loading is

$$f_s = f_{se} + f_{sa}$$

$$\therefore\quad f_s = \frac{P}{A_s} + \left[f_{P1} + (f_{P2} - f_{P1})\frac{kd}{D}\right]\frac{n(d\text{-}h)}{(kd\text{-}h)} \qquad \ldots\ldots (8)$$

For equilibrium in Fig. 1(d) $\Sigma H = 0$.

$$\therefore\quad (f_{L1} + f_{P1})\frac{kd.b}{2} = \frac{f'_{mr}}{2}b(qd\text{-}kd) + f_sA_s$$

Substituting Eq. (5), (6) and (8) in the above,

$$\left(\left[f_{P1} + (f_{P2} - f_{P1})\frac{kd}{D}\right]\frac{h}{kd\text{-}h} + f_{P1}\right)\frac{kdb}{2}$$

$$= \frac{f'_{mr}}{2}b\left[\frac{f'_{mr}\,kd}{\dfrac{h}{kd\text{-}h}\left[f_{P1} + (f_{P2} - f_{P1})\dfrac{kd}{D}\right] + f_{P1}}\right]$$

$$+ P + A_s\left[f_{P1} + (f_{P2} - f_{P1})\frac{kd}{D}\right]\frac{n(d\text{-}h)}{(kd\text{-}h)} \qquad \ldots\ldots (9)$$

In a particular beam the values of f_{P1}, f_{P2}, f'_{mr}, b, d, D, A_s, P and n are constant hence for various values of h, kd can be evaluated from Eq. (9).

Considering Fig. 1(d) and equating the external bending moment M to the internal moment of resistance.

$$M = (f_{L1} + f_{P1})\, kd\, b \frac{kd}{3} + f'_{mr}\,(qd\text{-}kd)b\,\frac{(qd\text{-}kd)}{3} + f_s A_s\,(d\text{-}kd)$$

and substituting values for f_{L1}, qd and f_s from Eq. (5), (6) and (8)

$$M = \left[f_{P1} + \frac{h}{(kd\text{-}h)}\,(f_{P1} + (f_{P2} - f_{P1})\frac{kd}{D})\right]\frac{b(kd)^2}{3}$$

$$+ \frac{f'_{mr}}{3}\, b \left[\frac{f'_{mr}\, kd}{\frac{h}{kd\text{-}h}\,(f_{P1} + (f_{P2} - f_{P1})\frac{kd}{D}) + f_{P1}}\right]^2$$

$$+ \left[\frac{P}{A_s} + (f_{P1} + (f_{P2} - f_{P1})\frac{kd}{D})\frac{n(d\text{-}h)}{kd\text{-}h}\right] A_s(d\text{-}kd) \qquad \ldots . (10)$$

For a particular beam Eq. (10) expresses M in terms of kd and h as f_{P1}, f_{P2}, f'_{mr}, b, d, D, A_s, P and n are constant. Thus M is given as a function of h by Eq. (9) and (10). By assuming arbitrary values of h, kd, can be calculated from Eq. (9) and then by substitution in Eq. (10), M is determined.

Stress determination in normal pre-tensioned beams.

A normal pretensioned rectangular beam is one in which the beam cross-section is constant, the pretensioned wires are straight and located at the lower third-point of the beam cross-section. The term normal prestress refers to a beam having a concrete stress distribution due to prestress of from approximately zero at the top to 2000 psi at the bottom fibre respectively. When the usual allowable steel prestress is employed this results in a small steel area of the order of 1% to 2% and its effect on the cross-sectional properties is usually small. If the steel area A_s is neglected it follows from Eq. (3) and (4) that $f_{P1} = 0$ and $f_{P2} = \frac{2P}{bD}$. By substituting these values, Eq. (9) and (10) become,

$$(kd)^2\,(Ph^2 - \frac{b^2d^4f'^2_{mr}}{4P}) + (kd)\,(\frac{b^2D^4f'^2_{mr}h}{2P} - PD^2h - \frac{2\,PnA_sh(d\text{-}h)}{b}$$

$$+ (PD^2h^2 - \frac{b_2D^4f'^2_{mr}h^2}{4P}) = 0 \qquad \ldots . (11)$$

$$\text{and } M = \frac{2Ph(kd)^3}{3D^2(kd\text{-}h)} + \frac{f'^3_{mr}\, b^3D^4(kd\text{-}h)^2}{12\,P^2\,h^2} + P(d\text{-}kd) + \frac{2PnA_s(kd)(d\text{-}h)(d\text{-}kd)}{bD^2\,(kd\text{-}h)} \qquad \ldots . (12)$$

The extreme fibre compression stress $fc_1 = f_{P1} + f_{L1}$ and by substituting for f_{L1} from Eq. (5).

$$f_{c1} = \frac{2Ph(kd)}{bD^2(kd-h)} \quad \ldots\ldots (13)$$

Substituting $f_{P1} = 0$ and $f_{P2} = \frac{2P}{bD}$ in Eq. (8) for the steel stress

$$f_s = \frac{P}{A_s} + \frac{2Pn\ (kd)\ (d-h)}{bD^2\ (kd-h)} \quad \ldots\ldots (14)$$

For the general case Eq. (9) and (10) can be solved and back substituted in Eq. (3), (5) and (8), to obtain the extreme compression fibre stress f_{c1} and the steel stress f_s. In the case where the pretension force is located at third-point of the cross-section Eq. (11) and (12) are first solved, then f_{c1} and f_s determined from Eq. (13) and (14). in both cases it is prudent to plot graphs of kd, M, f_{c1} and f_s against h to detect any error in the calculated values, and graphs covering the range of beams tested are shown in Figs. 2, 3 and 4. As the initial concrete prestress for these beams was either 30%, 100% or 300% of that normally used it is apparent that a large range of prestress was encompassed in the experimental verification of the proposed theory.

Determination of deflection

The prestress force creates stresses f_{P1} and f_{se} which are usually applied in a direction to cause upward deflection. Generally the deflection due to an external bending moment M is downwards and is determined by solving the equation

$$\frac{d^2y}{dx^2} = \frac{1}{dE_s}(f_{sa} + nf_{L1}) \quad \ldots\ldots (15)$$

or in terms of f_s and f_{c1}

$$\frac{d^2y}{dx^2} = \frac{1}{dE_s}((f_s - f_{se}) + n\ (f_{c1}\ f_{P1})) \quad \ldots\ldots (16)$$

If the bending moment diagram is known, the graph of

$\frac{1}{dE_s}((f_s - f_{se}) + n\ (f_{c1} - f_{P1}))$ can be plotted. As this function varies non-linearly with the applied bending moment, then in the general case it may have to be integrated either graphically or numerically.

Relation between theories for deflection before and after cracking

The general theory for deflection in the cracked range can be used to predict reflection prior to cracking provided certain modifications to the theory are made. The combined stress distribution for an uncracked cross-section is shown in fig. 5.

The stress distribution below the bottom fibre is imaginary and if the terms relating to this portion are neglected, the resultant equations predict the deflection of an uncracked section.

To simplify the algebra for comparison with Navier's equation, the particular loading that causes kd to equal D as shown in fig. 6 can be considered.

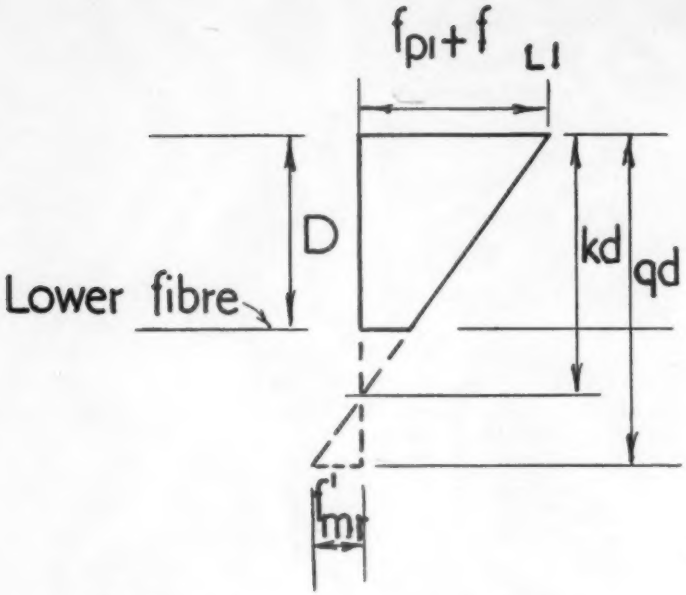


Fig. 5 — A typical stress distribution prior to cracking

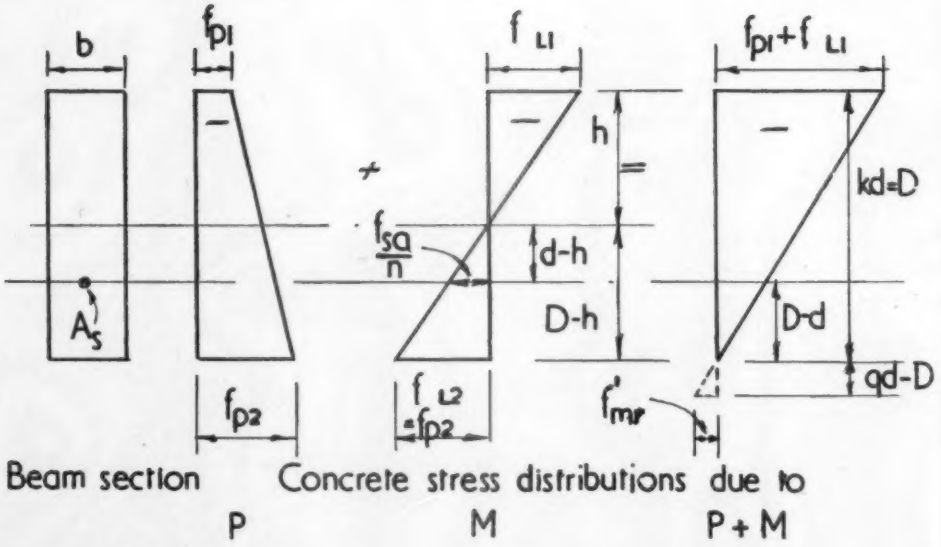


Fig. 6

Eq. (5), (8) and (15) can now be used to derive the deflection equation $\frac{d^2y}{dx^2} = \frac{n\ f_{P2}}{E_s(D-h)}$ which is the same as Navier's equation $\frac{d^2y}{dx^2} = \frac{M}{E_c I_D}$.

It is apparent that both theories predict the same deflection prior to cracking. For simplicity, Navier's equation should be used up to the theoretical cracking bending moment M_c and the proposed theory for the cracked range thereafter. M_c is determined from the equation,

$$M_c = (f_{P2} + f'_{mr})Z_2 \quad \ldots . (17)$$

EXPERIMENTAL DETAILS

Beam Manufacture

In addition to developing a theory for deflection, the cracking characteristics and the ultimate bending moment were studied. During the investigation, the concrete cylinder strength f'_c was varied from 4330 to 9890 psi, the steel stress prior to transfer from zero to 155,000 psi, the steel porportion from 0.00125 to 0.031 and the concrete lower fibre prestress at transfer from zero to 6000 psi. These ranges were selected not only to cover all cases likely to occur in the practical design of non-prestressed and prestressed beams subjected to flexure failure, but also to provide a basis for theory well beyond this range.

The concrete consisted of high early strength portland cement, coarse river sand and ¾″ maximum sized round river gravel and was well vibrated into steel channel moulds. The concrete was generally of zero slump and the measured cement content varied from 5 to 12 bags per cubic yard. The control cylinders were vibrated in two layers, cured under the same conditions as the beams and tested at the same age. The beams were pretensioned in a 25′0″ pre-tensioning bed, the prestress being transferred to the beams at 4 days, after which both beams and cylinders were immersed in water for 30 days. The beams and cylinders were then removed from the water and air dried for 2 days to enable the necessary instrumentation before testing to destruction.

The steel prestress prior to transfer was measured to 1% accuracy by means of a Gerard Extensometer. The elastic, creep and shrinkage strains in the concrete were measured by means of a Whittemore Deformeter between brass targets attached to the concrete. Hence the effective steel prestress at the time of beam test was known to about 2% accuracy.

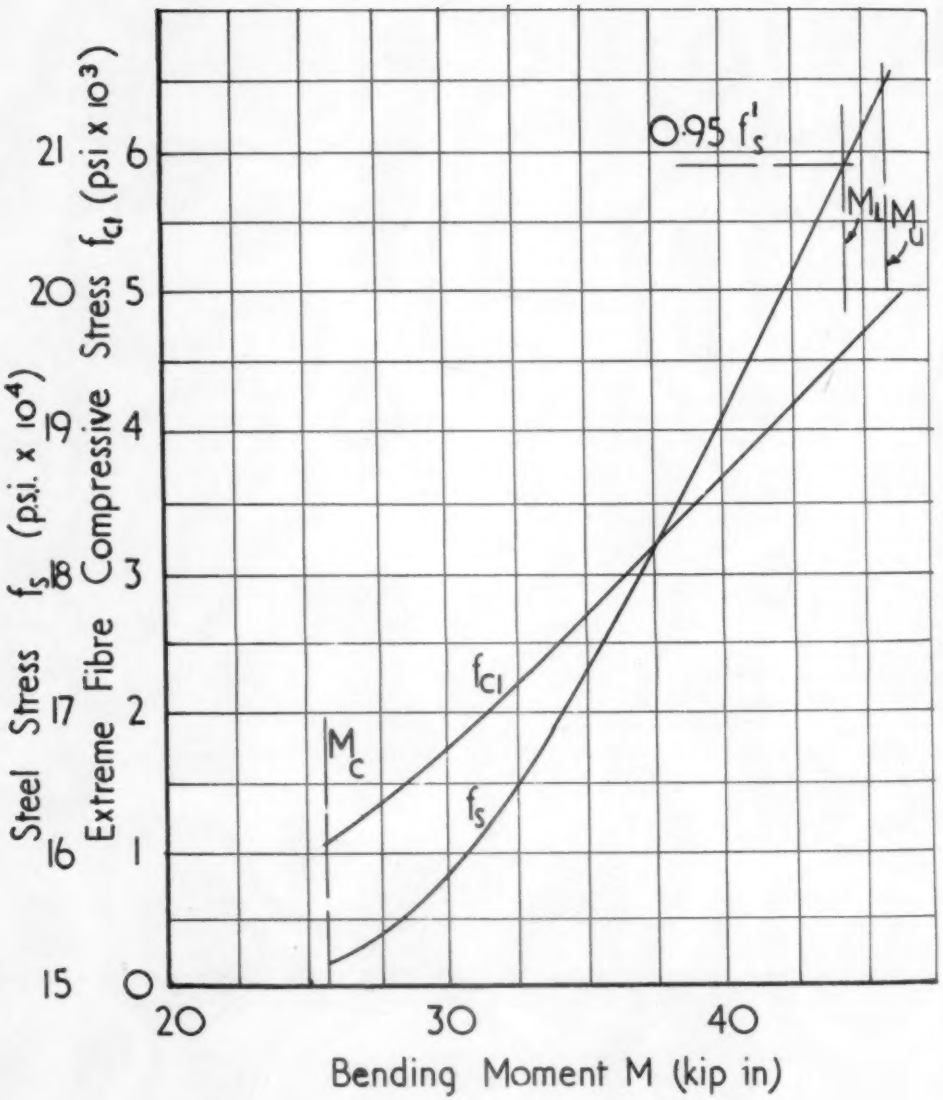
Testing details

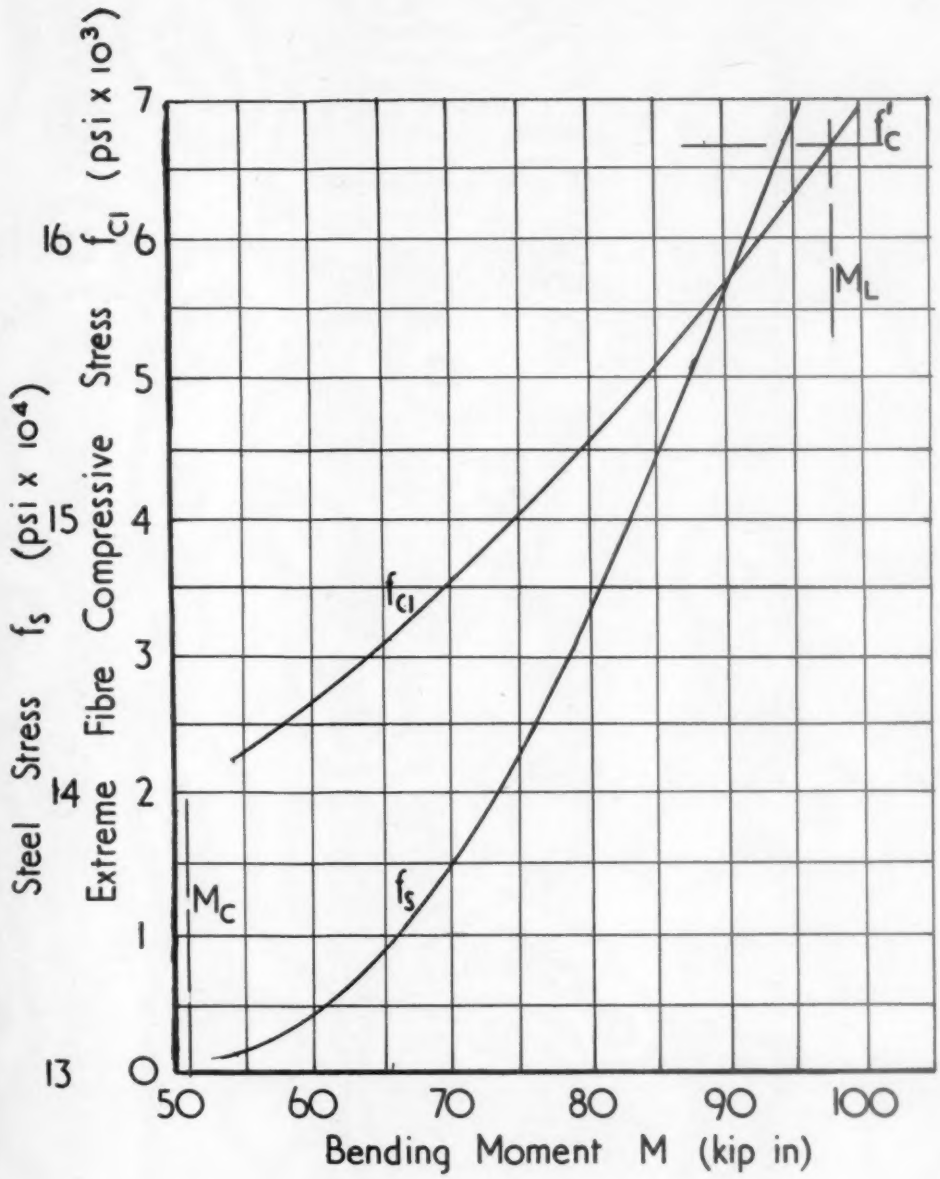
The 6″x4″x10′0″ pretensioned beams were third-point loaded over a 9′0″ span and deflections recorded under both load points and at the mid-span position. Mercer dial gauges of least count 0.0001″ attached to a light steel angle resting on the beam supports were used to observe the beam deflection. The load was applied at a slow rate, in 200 lb. increments, with pauses for observations at each loading so that the necessary readings could be observed. The load was increased in continuous increments from zero load up to the collapse value in one complete operation, over a period of about one hour.

The modulus of elasticity of the concrete was obtained from concrete control cylinders according to British Standard 1881:1952 "Methods of Testing Concrete" and the modulus of elasticity of the steel according to British Standard 18:1956 "Tensile Testing of Metals."

Beam properties

For a beam of given dimensions, the variable with the greatest effect on deflection in the cracked range is the prestress force. In view of this the three sets of beams used for comparing the observed with the calculated deflections were selected to cover the largest possible range of prestress force. Only the details relating to these beams are included in Table 1.


Steel Stress f_s (p.s.i. x 10^4)
Extreme Fibre Compressive Stress f_{cl} (psi x 10^3)
21
20
19
18
17
16
15
6
5
4
3
2
1
0
20
30
40
Bending Moment M (kip in)
0.95 f'_s
M_L
M_U
M_C
f_{cl}
f_s


Steel Stress f_s (psi x 10^4)
16
15
14
13
Extreme Fibre Compressive Stress f_{cl} (psi x 10^3)
7
6
5
4
3
2
1
0
50
60
70
80
90
100
Bending Moment M (kip in)
f_{cl}
f_s
M_c
M_L
f'_c

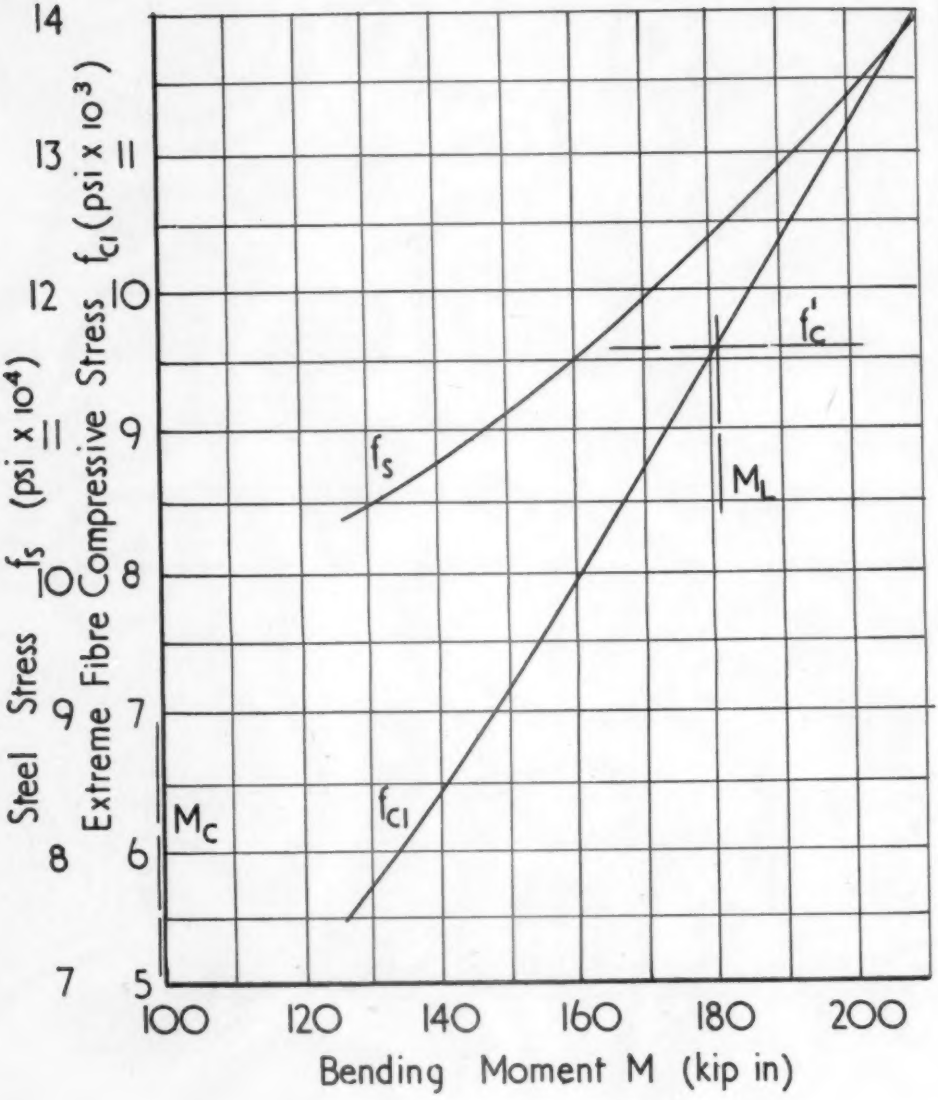

Steel Stress $\frac{f_s}{10}$ (psi x 10^4)
Extreme Fibre Compressive Stress f_{c1} (psi x 10^3)
14
13
12
11
10
9
8
7
6
5
f_s
f_{c1}
f'_c
M_L
M_c
100
120
140
160
180
200
Bending Moment M (kip in)

TABLE 1. BEAM DETAILS

Beam Properties		Beam Numbers				
		30% normal prestress		Normal Prestress		300% normal prestress
Variable	Units	1/1	1/2	2/1	2/2	3/1
b	in.	4	4	4	4	4
d	in.	4	4	4	4	4
D	in.	6	6	6	6	6
A_s	in.^{2}	0.06	0.06	0.179	0.179	0.495
P	lb.	9000	9000	22,500	22,500	46,000
f'_{mr}	psi	320	320	270	270	320
f'_c	psi	9890	9890	6,670	6670	9600
f'_s	psix10^3	220	220	220	220	217
M_c(Theor.)	lb.in x10^3	25.7	25.7	52.5	52.5	99.6
M_u (Obs.)	lb.in x10^3	46.0	45.4	122.0	120.3	218.1
E_c	psix10^6	4.4	4.4	4.22	4.22	4.4
E_s	psix10^6	28.2	28.2	28.2	28.2	28.2
n		6.4	6.4	6.7	6.7	6.4

COMPARISON OF RESULTS.

Figs. 2, 3, and 4 show kd, f_{c1}, f_s and M as functions of h for the three beams considered. From these graphs, Figs. 7, 8, and 9 are plotted showing f_{c1} and f_s in terms of M for the same three cases. The limits within which the proposed equations apply are indicated by the bending moment values M_c and M_L. The lower limit M_c is the theoretical cracking bending moment determined from Eq. (17). The upper limit M_L is the bending moment at the limit of stress linearity and is equal to the least value of bending moment at which either f_c, first attains the value f'_c or f_s equals $f_s y$. In this investigation $f_s y$ for the high tensile strength steel is taken as 0.95 f'_s.

It should be noted that in this investigation the effect of creep on deflection has been neglected as the test period was of short duration. Additional investigation is underway to enable modification of the proposed deflection theory for beams subjected to long term loading.

Theoretical deflection of the beams tested.

Prior to cracking the whole cross-section is operative for the total length of a beam consequently the moment of inertia is constant and the function $\frac{1}{dE_s}\left((f_s - f_{se}) + n(f_{c1} - f_{P1})\right)$ varies linearly with applied bending moment. When the theoretical bending moment M_c is exceeded cracking will commence, but while the applied bending moment is uniform this function will also be uniform. For any other distribution of applied bending moment, greater than M_c, the function $\frac{1}{dE_s}\left((f_s - f_{se}) + n(f_{c1} - f_{P1})\right)$ will vary non-linearly.

The beams tested were symmetrically loaded, consequently once cracking commences the $\frac{1}{dE_s}\left((f_s - f_{se}) + n(f_{c1} - f_{P1})\right)$ diagram will consist of a linear, a non-linear and a uniform portion as shown in Fig.10.

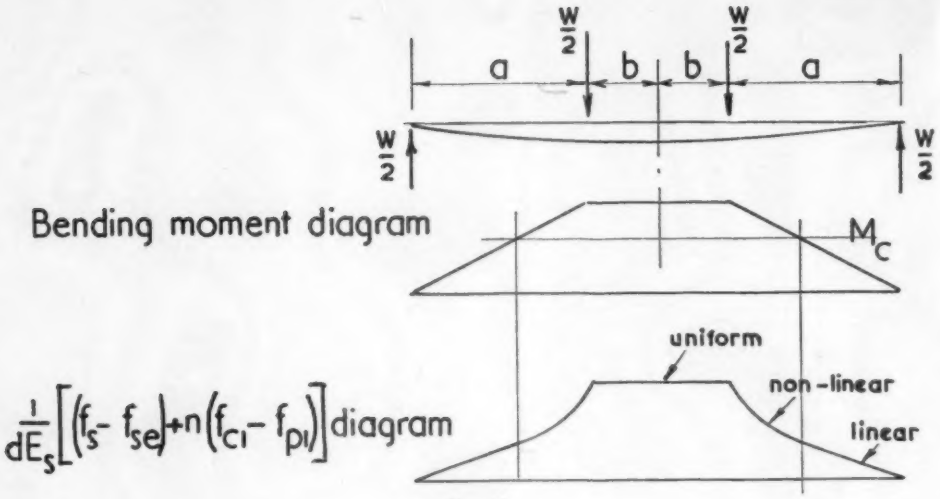


Fig. 10

For a particular loading, the $\frac{1}{dE_s}((f_s - f_{se}) + n(f_{c1} - f_{P1}))$ diagram is integrated graphically to obtain the deflection and the theoretical moment deflection curves for the beams tested are shown in Figs. 11, 12 and 13. It can be seen from these figures that the correlation between the theoretical and observed deflection curves supports the proposed theory over a large range of prestress force.

CONCLUSIONS

1. Rigorous equations are derived relating kd, M, f_{c1}, f_s to h for a rectangular prestressed beam, after cracking has commenced.
2. These equations are simplified for the case where the prestressing force is placed at the third-point of the cross-section and the effect of the small steel area on the beam properties is neglected.
3. A theory for deflection is proposed based on the variation of f_{c1} and f_s along a beam.
4. When the bending moment is uniform the $\frac{1}{dE_s}((f_s - f_{se}) + n(f_{c1} - f_{P1}))$ diagram will be constant. The deflection is then evaluated analytically.
5. When the bending moment is variable the $\frac{1}{dE_s}((f_s - f_{se}) + n(f_{c1} - f_{P1}))$ diagram will be complicated and difficult to integrate. The simplest method of solution will probably be graphical integration, introducing the boundary conditions to evaluate the constants of integration.
6. Good correlation was obtained between the theoretical and observed deflection curves of all beams tested.

ACKNOWLEDGMENTS

This paper is based on an unpublished thesis submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy at The University of New South Wales.

Appreciation is expressed to the research team of the School of Civil Engineering Structural Research Laboratory for their untiring assistance associated with the physical testing.

The Author wishes to thank Professor A. S. Hall for his helpful comments in the preparation of this paper and for his criticism of the draft copy.

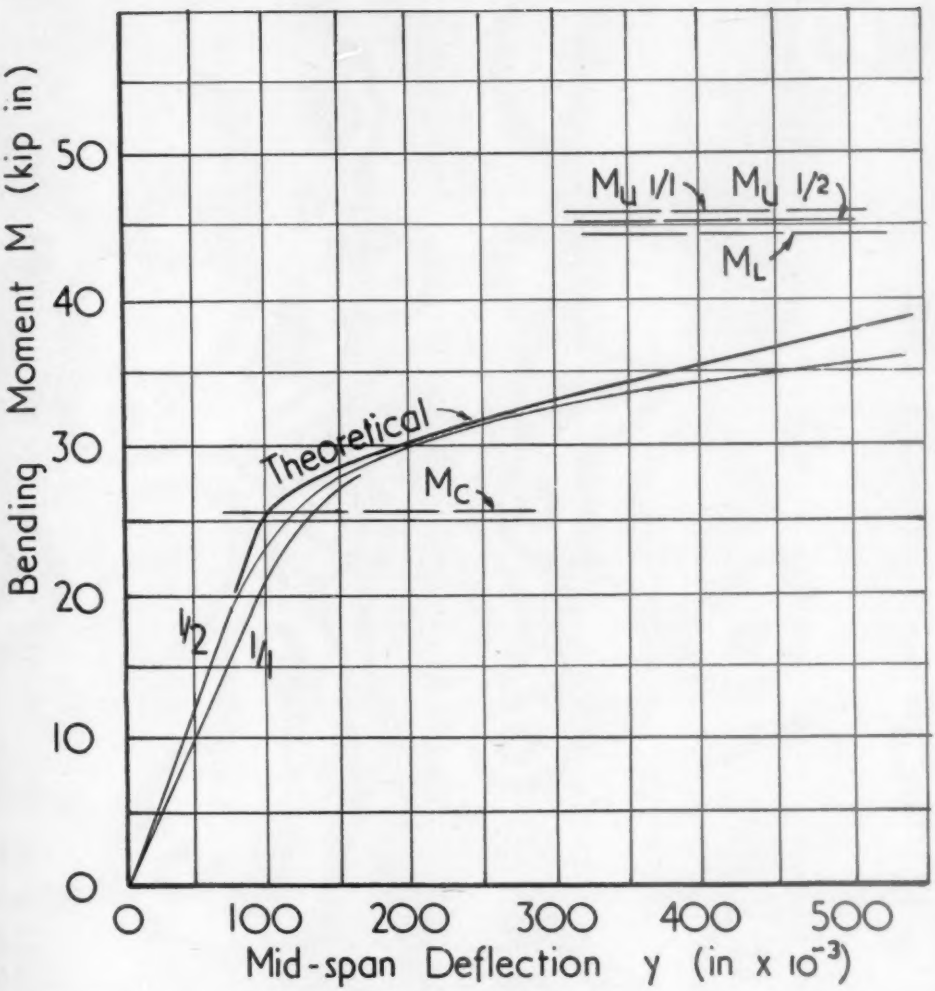

Bending Moment M (kip in)
0
10
20
30
40
50
0
100
200
300
400
500
Mid-span Deflection y (in x 10⁻³)
Theoretical
Mc
ML
MU 1/1
MU 1/2
1/2
1/1

Fig. 11

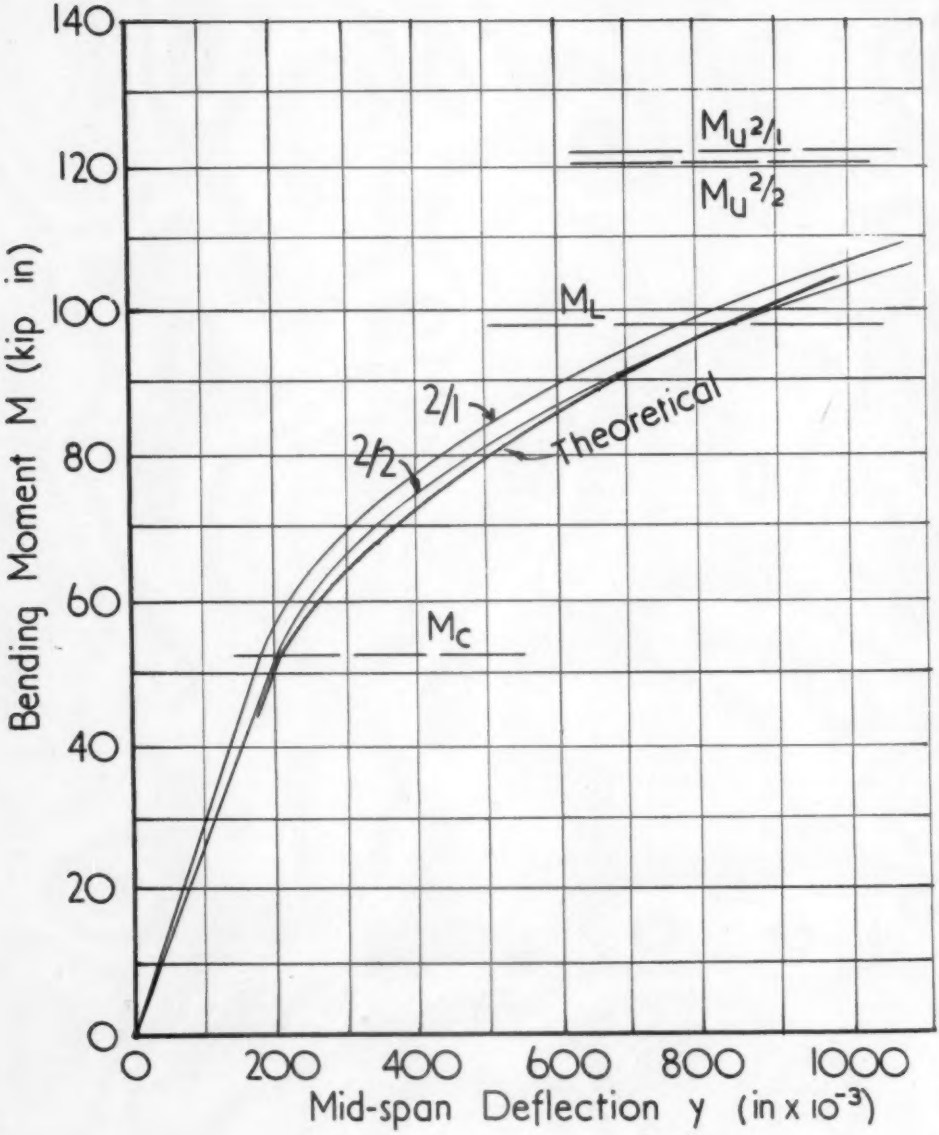

140
120
100
80
60
40
20
0
Bending Moment M (kip in)
0
200
400
600
800
1000
Mid-span Deflection y (in x 10⁻³)
$M_U{}^{2}/_{1}$
$M_U{}^{2}/_{2}$
M_L
M_C
2/1
2/2
Theoretical

Fig. 12

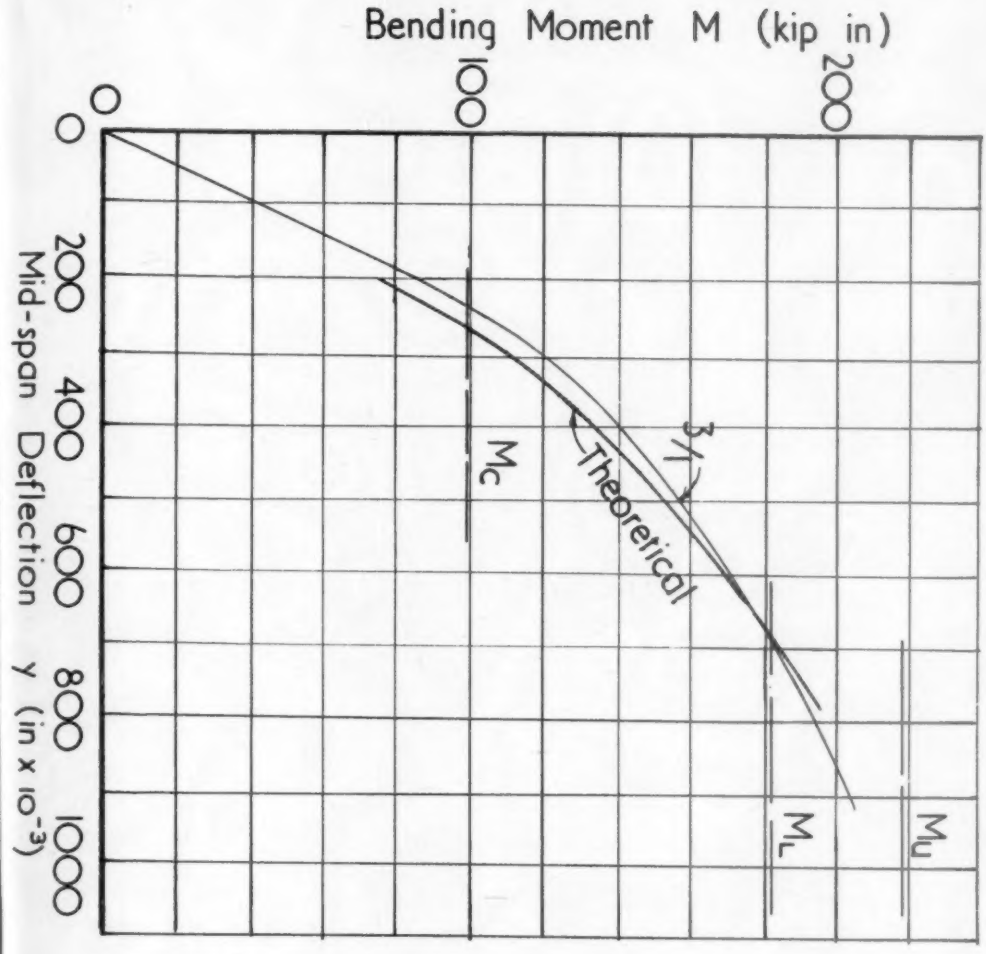

Bending Moment M (kip in)
0
100
200
0
200
400
600
800
1000
Mid-span Deflection y (in x 10⁻³)
Mc
Theoretical
3/1
ML
Mu

Fig. 13

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APPENDIX.

Theoretical stresses in a particular beam.

To explain the theory and to reduce the algebra involved, a particular beam number 2/1 subjected to normal prestress is examined. As this beam is rectangular with the prestress force located at the third-point of the cross-section then $f_{P1} = 0$ and $f_{P2} = \frac{bD}{2P}$.

Eq. (11), (12), (13) and (14) can now be further simplified before calculating the beam stresses f_s and f_{c1}.

Substituting in Eq. (11)

$$(kd)^2 (22500h^2 - 16796.16) + kd (13492.12\, h^2 - 830376h) + 793\,203.84\, h^2 = 0 \quad \ldots (11a)$$

Substituting in Eq. (12)

$$M = \frac{416.67h(kd)^3}{(kd-h)} + \frac{268.74(kd-h)^2}{h^2} + 22500\,(4-kd) + \frac{374.78(kd)(4-h)(4-kd)}{(kd-h)} \quad \ldots (12a)$$

Substituting in Eq. (13) for the extreme compressive stress

$$f_{c1} = \frac{312.5h\,(kd)}{(kd-h)} \quad \ldots (13a)$$

Substituting in Eq. (14) for the steel stress

$$f_s = 125698.3 + 2093.75\,\frac{kd(d-h)}{(kd-h)} \quad \ldots (14a)$$

Having reduced the algebra particular values of h are assumed and kd, M, f_{c1} and f_s calculated from Eq. (11a), (12a), (13a) and (14a). Such values included in Table 2 are used to plot Figs. 3 and 8.

TABLE 2 — Theoretical values of variables related to h.

h in.	kd in.	k	M lb. in.	f_{c1} psi	f_s psi
3.0	4.875	1.219	56 804.8	2437.6	131,142.0
2.8	3.891	0.973	65 666.5	3120.9	134,659.0
2.5	3.078	0.769	76 046.1	4158.0	142,413.0
2.2	2.508	0.627	88 663.0	5591.9	156,351.0
2.1	2.346	0.587	94 337.0	6253.8	163,608.0
2.0	2.194	0.549	101 340.0	7071.6	173,078.0

Theoretical deflection of a particular beam.

Calculations for deflection of the previous beam are made using data from Tables 1 and 2. The results are listed in Table 3.

TABLE 3.

M lb. in.	$\frac{1}{dE_s}((f_s - f_{se}) + n(f_{c1} - f_{P1}))$ in^{-1}
52 500	1.73×10^{-4}
65 666	2.64×10^{-4}
76 046	3.94×10^{-4}

When the maximum bending moment is 76046lb. in., the bending moment and the $\frac{1}{dE_s}((f_s - f_{se}) + n(f_{c1} - f_{P1}))$ diagrams are as shown in Fig. 14.

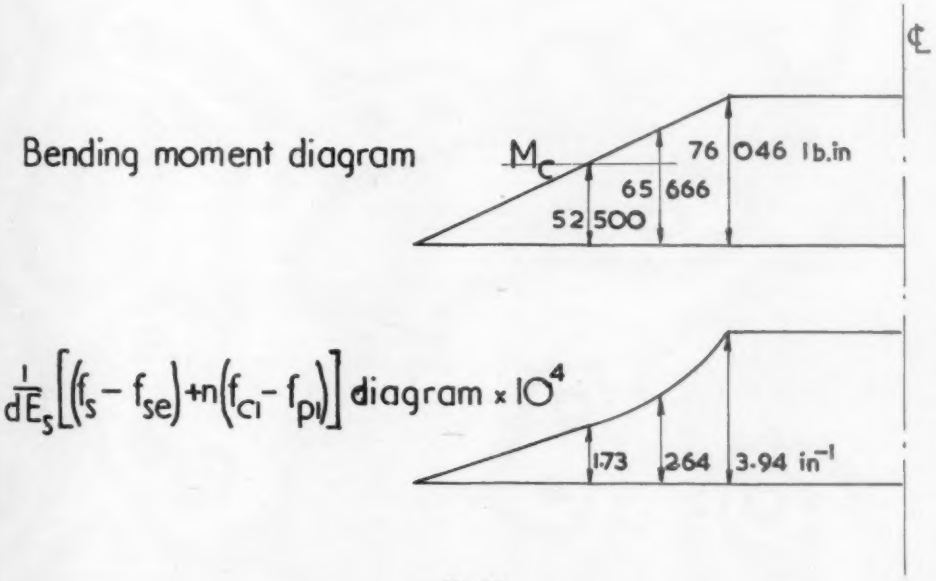

Fig. 14

Graphical integration of the $\frac{1}{dE_s}((f_s - f_{se}) + n(f_{c1} - f_{P1}))$ diagram gives the deflection due to this particular load. Further values are similarly obtained and the moment-deflection curve plotted as shown in Fig. 12.

PREPARATION AND DURABILITY TESTING OF PRETENSIONED PRESTRESSED CONCRETE

By

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Directed by

THE ILLINOIS PRESTRESSED CONCRETE ASSOCIATION
(Under personal charge of J. O. Whitlock)

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INTRODUCTION

Due to the high cost of labor, it has been only within the past decade that prestressed concrete has become a competitive construction material in the United States. The material is now being used extensively in sections of the country where the effect of freezing and thawing action is an important factor with which to reckon. Although concrete under compression was generally believed to exhibit excellent resistance to weathering action, a number of leading Illinois prestressed concrete companies initiated tests in order to more fully determine its durability.

The Durability Test Program was undertaken by the Illinois Prestressed Concrete Association in collaboration with the Prestressed Concrete Institute. In addition, the Corps of Engineers is preparing to run brine tests on prestressed beams loaded in flexure. These beams are set in the tide wash along the coast of Maine, and will probably be under test for two years.

The investigation reported herein is fundamentally concerned with the durability of prestressed concrete in regard to freezing and thawing action. Some provisions were made to study the creep in the loaded concrete specimens undergoing the tests.

Work on this particular project began during the summer of 1958 at Midwest Prestressed Concrete Company's plant at Springfield, Illinois. A group of test specimens were programmed in an attempt to determine the effect of three primary variables on the durability of 200 prestressed concrete beams 3 by 4 in. in cross section by 32 in. in length. Programming was conducted in a manner to include the following variables: Level of stress in the concrete, method of curing (moist and steam), and type of cement (Type I and Type III). The test specimens were prepared in five series of 40 beams each. The test results obtained from any one series of 40 beams should then be very suggestive of the results to be obtained by testing the entire group of 200 beams. Consequently, it would be feasible to test only two or three series and be able to obtain an indication of the durability of the specimens. Compressive strengths of molded concrete cylinders were obtained at frequent intervals during the preparation of each series.

To facilitate molding of the beam specimens, a special prestressing bed was developed. It is discussed herein because of its uniqueness and importance in the preparation of all test specimens used in the investigation.

A lengthy discussion concerning preparation of the test specimens is included in the text, since the procedure employed is believed to be unique and may therefore be of interest to the reader.

The testing procedures conformed as nearly as practicable to ASTM specifications. All beams tested were subjected to a minimum of 400 fast cycles of freezing and thawing. The dynamic modulus of elasticity, weight, and length change over a 10-in. gauge length were periodically recorded for each beam tested as an aid in determining the effects of the variables on the durability.

Time permitted the testing of only two series of specimens; therefore, results presented, analyzed, and discussed in the investigation are concerned primarily with these two series.

PROGRAMMING OF TEST SPECIMENS

Variables

In an attempt to determine how certain factors affect durability of prestressed concrete, a group of test beams and cylinders was programmed to include three major variables consisting of:

1. Level of stress in the concrete.
2. Method of curing.
3. The type of cement.

The variables were introduced so that their effects could be studied on the basis of specimens made within the shortest possible period of time. It has been found that the variability, or so-called error of testing, is much greater for mixtures made over an extended period than for mixtures produced over a short interval, and may be so great that it tends to obscure the effort of the variables which are the subject of the study.

Level of stress in the concrete. The stresses in the concrete beam specimens were programmed to vary by 500 psi increments beginning with zero psi and ending with 2000 psi. Such a range of stresses should provide an indication of what effect different levels of stress have on the durability of the specimens. It should be understood that while undergoing tests the beams were not subjected to eccentric loads which could produce flexure and thus possible cracking of the concrete. The beams were prestressed by the pretensioning method in a manner such that the compressive stress distribution within the specimen should be symmetric with respect to the horizontal and vertical axes of the beam.

Method of curing. In acutal prestressed construction practice, it is not economically practical to have a prestressing bed occupied with curing concrete for any lengthy period. Of course, the concrete need not attain its full design strength, which is assumed to require 28 days, but must be strong enough to transfer the force in the steel to the concrete through bond, and also to carry any stresses caused by handling. Since the use of steam curing is one of the means used to attain a reasonably high strength concrete in a comparatively short period of time, a comparison between moist and steam curing methods generally in use was included in the tests. One-half of the specimens programmed for the tests were moist cured, whereas the remainder received a combination of steam and moist curing.

The type of cement. High-early-strength cement may be employed along with steam and/or moist curing in developing the high strength concrete required for removal of the concrete from the prestressing bed. Type I and Type III portland cements were used in an equal number of specimens to compare their effects on the durability and creep of the loaded concrete.

Constants

The factors which were held as constant as possible throughout the investigation may be itemized as follows.

1. Dimensions of specimens.
2. Comparative specimens from each series and batch of concrete.
3. Mixture design.
4. Curing period.

Dimensions of specimens. Since the pretensioning method of prestressing was utilized, the specimens had to have a sufficient transmission or transfer length to develop the required stress through bond between concrete and tensioning strand. Conversely, the beams had to be of a size to accommodate the freeze and thaw apparatus, and light enough in weight to be readily handled by hand. The limitations of the mechanical freeze and thaw apparatus necessitated that the specimens be made no larger than 3- by 4-in. in cross section by 32 in. in length. Two ⅜-in. diameter strands were positioned relative to the beam as shown in Plate I. To insure that the beam specimens atained the proper stress over a greater part of their length, cable clamps were attached to each strand at the ends of the specimens to function as bond increasers, thus reducing the transfer length.

The specimens prepared for compressive strength studies were 6- by 12-in. cylinders.

Comparative specimens from each series and batch of concrete. The group of test specimens was programmed into five series, each series containing 40 beams, thus giving a total of 200 test beams. Each series possessed the same variables and was placed on days approximately at one week intervals. Theoretically, the test results obtained from any one series of 40 beams should be indicative of the results to be gained by testing the entire group

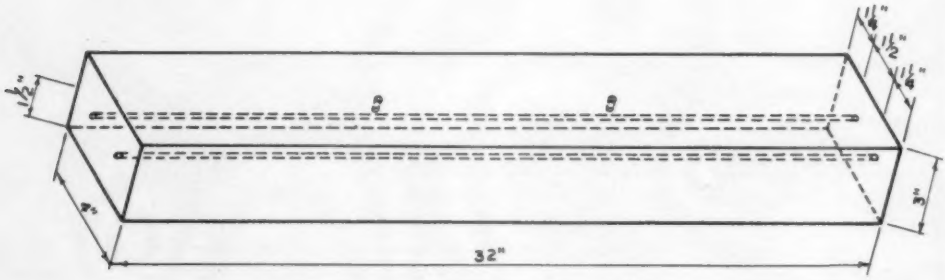


PLATE I

Isometric view of beam specimen showing positioning of steel strand. Bond increasers attached to each strand at ends of specimen are not shown in view.

of 200 beams. Accordingly, it would be feasible to test only those series which time permitted, and be able to obtain an indication of the durability of the specimens.

The table 1 is a tabulation showing the programming and number of beams in one series.

Table 1. Programming of beam specimens for each series.

Stress	Type I cement		Type III cement	
psi	Moist	Steam	Moist	Steam
0	2	2	2	2
500	2	2	2	2
1000	2	2	2	2
1500	2	2	2	2
2000	2	2	2	2

It shall be noted that each series contains two beams representing each condition to be studied; after five identical pours, there would exist ten specimens for each condition to be investigated in the durability tests. For instance, upon completion of molding five series of specimens, ten beams are acquired having 1000 psi stress, Type III cement, and moist curing; also ten beams having 1000 psi concrete, Type III cement, and steam curing, etc.

Since the investigation was to extend over a prolonged period, ten zero psi concrete beams obtained for each cement and type of curing were included as standards of comparison. These particular beam specimens are strictly comparable to each set of ten beams representing any other condition.

The test beams are not of the same age; however, this should not really matter, since the tests were not started at an early age. The specimens' attainment of considerable age before the testing operation began should minimize the effect of differences in age. Actually, all specimens may be considered as being made on the same day; that is, on a day that represents the average of the five weeks of pour.

Table 2 illustrates the programming of the test cylinders used in the compressive strength investigations.

Table 2. Programming of cylinder specimens for each series showing breaking plan.

Age	Type I cement		Type III cement	
days	Moist	Steam	Moist	Steam
1	2	2	2	2
2	2	2	2	2
3	2	2	2	2
4	2	2	2	2
7	2	2	2	2
14	2	2	2	2
28	2	2	2	2

Eighteen cylinders were molded for each type of cement and method of curving giving 72 cylinders per series or a total of 360 for the five series poured. Cylinder breaks were made at 1, 2, 3, 4, 7, 14, and 28-day intervals for each series poured to determine the effect of type of cement and curing method on the compressive strength of the concrete.

Mixture design. The design of the mix used for all durability test specimens and other pertinent information concerning the mix may be found in the appendix.

Types I and III portland cements were obtained from the same manufacturer. Cement content was maintained at 7 bags per cubic yard. This is a figure comparable with that used in prestressed construction practice.

The coarse aggregate was a crushed limestone with 1 in. maximum size. A fineness modulus of 2.47 was recorded for the sand used as fine aggregate. Both aggregates were stockpiled and weighed to obtain the desired proportions of 1¾: 3.

The only mixer readily available to mix the desired ¾ cubic yard batch was a 4 cubic yard truck mixer. A scheme was devised whereby this mixer could be satisfactorily used throughout the test.

An attempt to maintain the entrained air content and slump of the fresh concrete at 4 per cent and 1 in., respectively, was met with limited success.

Curing period. All specimens were accorded the same curing period of 3½ days. Steam was applied to the appropriate specimens for 13 hours, and then the beams and cylinders were moist cured to the completion of the curing period. The remaining specimens, of course, were subjected to moist curing throughout the curing interval.

DEVELOPMENT OF PRESTRESSING BED AND COMPONENTS

Construction of Prestressing Bed

To facilitate molding of the test specimens, a special prestressing bed was designed and constructed. The bed had to withstand the pretensioning loads, accommodate 40 beams plus 72 cylinders arranged in accordance with the programming, and provide for steam and moist curing specimens.

The bed constructed to meet these requirements is illustrated in Plate II. The reinforced concrete bed, weighing approximately eight tons, was cast upside down and later inverted to its upright position. A bulkhead divided the interior into two compartments; one side for steam and the other side for moist curing. Styrofoam, 1½-in. thick, was sandwiched in the center bulkhead to provide insulation against heat transfer from the steam side to the moist compartment. Stuffing was placed in the strand portals prior to introducing the steam.

The floor of the bed and 3- by 2- by 3/16-in. angle iron with 3-in. legs spaced 4-in. back to back served as beam molds. Five lines of these molds were aranged in the longitudinal direction with room being left on the sides to store the cylinders during the curing period. The angle iron forms were fastened to the concrete bed by anchor screws with one side of the form left stationary while the other side was moveable to facilitate removal of beam specimens. The five lines of molds, one for each programmed stress level, contained eight beams each. Metal bulkheads, ¼-in. thick, were spaced at proper intervals to provide for the 32-in. beam length. Each bulkhead was perforated to permit passage of the prestressing steel, and was held in position by a metal pin and bolt inserted behind the plate (Fig 16, Appendix).

Steel strand was fixed at a position coincident with the horizontal axis of the beam by

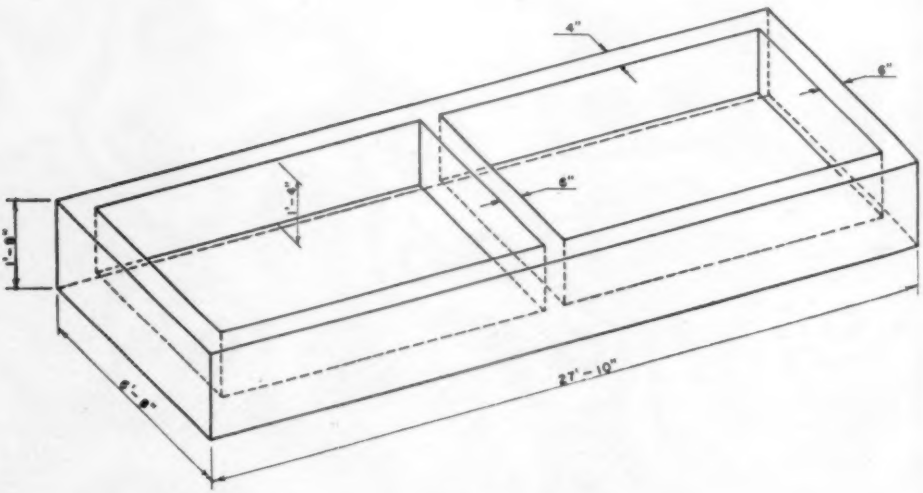


PLATE II

Isometric view of prestressing bed used in preparation of specimens. Details not shown.

Strand-Vises abutting against a steel plate and a strand guide system cast into an end concrete bulkhead. The other end bulkhead contained a similar plate and strand guide system; however, larger bearing plates were welded to the plate-guide system to provide a firm tensioning base for the jack.

The casting bed was located in a position subject to the warming rays of the sun. To prevent the freshly poured concrete beam specimens from setting rapidly because of being placed in warm molds, a tarpaulin canopy was erected several feet above the bed (Fig. 18, Appendix).

Method of Prestressing

The force for tensioning the steel strand was supplied by a specially developed screw-type jack which, when incorporated with a load cell, provided an accurate method of applying the required load to the strands. Since a force of 12,000 lb. per strand was required to yield 2000 psi prestressed concrete specimens, the jack was sturdily constructed. Plate III shows an exploded view of the jack components.

Each strand at the tensioning bulkhead was grasped by the jaws of a threaded Strand-Vise. The jacking stool, containing the jacking screw, was positioned against the bearing plate and the draw pin threaded into the Strand-Vise. A load-cell, used in determining the load applied to the strand, was then placed on the oposite end of the draw pin followed by a washer and bearing nut. The keeper washer was placed between load cell and jacking screw to eliminate

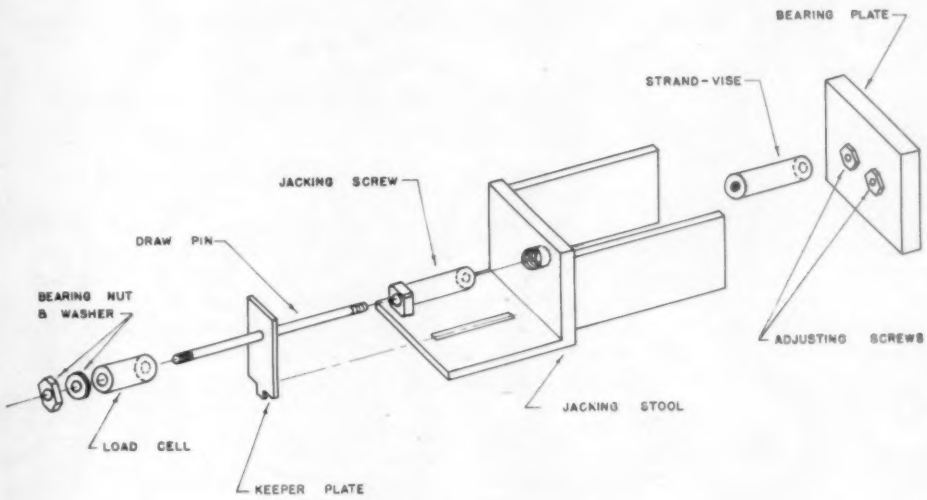


PLATE III

Exploded view of pretensioning components used in specially developed screw jack.

the torsional effect on the cell produced by the jacking screw during tensioning operations. The jacking screw was turned by a hand wrench until the proper load had been applied to the strand as evidenced by the load-cell strain readings recorded with a Baldwin SR-4 Strain Indicator (Figs. 13 and 14, Appendix). Metal shims, slotted to fit over ⅜-in. diameter strand, were placed between the Strand-Vise and adjusting screws located in the bearing plate in order maintain the requisite elongation. Remaining slack was removed by tightening the adjusting screws. The jacking screw was then turned in the opposite direction until the strand load was transferred to the bed, whereupon the jack was removed by unscrewing the draw pin from the Strand-Vise thus leaving one strand completely tensioned. This procedure was then repeated until all strands were tensioned the desired amount.

PREPARATION OF TEST SPECIMENS

Preparation of Bed

Prior to molding the specimens, the bed floor, molds, and plate bulkheads were coated with form oil. Paper covering was temporarily placed between the two sides of the molds while ⅜-in. diameter steel strand was threaded through the strand positioner guides imbedded in the tensioning end bulkhead. Eight plate bulkheads together with the 16 cable clamps bond increasers reuired for one line of molds in one compartment, were threaded on the two strands used in each of the five lines contained in the prestressing bed. The strands were then pulled through the center concrete bulkhead portals, and the remaining plate bulkheads and clamps were placed on the strands before the tensioning steel was passed through the strand positioner guides and anchored to the stationary end bulkhead with Strand-Vises. The strands were positioned coincident with the horizontal axis of the beam.

New tensioning forces were applied to the strands by a screw-type jack in the manner described on page 107. Strands in the center line molds were tensioned to produce 2000 psi prestressed concrete with zero psi, 1500 psi and 1000 psi, 500 psi, lines running on the opposite sides of the center line. Only tension large enough in magnitude to eliminate most of the sag in the 27 ft. length of steel was applied to the zero psi line.

Gauge plates were used to insure proper spacing of the metal forms relative to the strands after tensioning operations were completed. Plate bulkheads were positioned along the lines at 32-in. intervals with adequate space left between ends of beams to allow fastening the plate in place with a metal bolt and pin, and to allow eventual severing of the strands between beams. Preparation of the bed for molding one series of specimens was completed after bond increasers were fastened to each strand approximatly 1½-in. from the beam ends. (Figs. 15 and 16, Appendix)

Molding of Specimens

Two ¾ cu. yd. batches were required to mold each series containing 40 beams for durability testing and 72 cylinders for compressive strength studies. Type I cement was used in the first batch, and Type III in the last of the series pour.

The stockpiled aggregates were weighed to obtain the proper proportions and deposited, via wheelbarrow and ramp, into a cubic yard capacity bucket. A lift truck was utilized to deposit the contents of the bucket together with Type I cement into the 4 cu. yd. transit mixer used to mix the concrete throughout the tests. Water containing approximately 90 cu cm of Darex AEA was added uniformly to the mix, and the load mixed for 50 revolutions of the drum. The batch was slowly agitated during the molding of the specimens.

Slump tests and entrained air measurements, by the pressure method, were taken at the beginning, middle, and end of each batch pour. An attempt was made to maintain the entrained air content at 4 per cent. The addition of an amount of Darex AEA to produce this air content was not always accomplished with a high degree of accuracy as evidenced by the fact that the air content ranged from an average reading of 3.3 to 4.8 per cent. This, however, is well within the tolerance of plus or minus 1½ per cent allowed by most specifications. The average entrained air content for the 10 batches poured was 3.8 per cent. Originally, the water content was to be maintained at 4.3 gal. per bag of cement, but this was varied somewhat to increase workability of the concrete. Slump readings ranged from a batch average of ½-in. to one high reading of 2¼-in.

The concrete was shoveled into the forms and around the strands, and then was thoroughly vibrated into all void spaces with a hand vibrator. The top surface of the beams was hand troweled to a smooth, flat surface.

Coincident with casting the 20 beams containing Type I cement, 36 cylinders were molded in a nearby room and were later transferred to the bed for curing. The 6- by 12-in. fiber cylinder molds were filled in three lifts; each lift being vibrated with a small hand vibrator. The exposed surface was hand troweled smooth and flat.

When the first batch had been poured, the mixer was cleaned, and another ¾ cu. yd. load was prepared using high-early-strength portland cement. After the initial air and slump

tests were taken the remaining 10 beam molds in each compartment of the bed were filled with concrete, vibrated, and hand troweled. Likewise, 36 cylinders containing the Type III cement were molded and then moved to the bed for curing. An average time of 40 minutes elapsed between the initial and final air and slump tests made on any one batch. An additional 60 minutes were needed to prepare the batch containing Type III cement.

The insertion of wooden blocks between the tensioning steel helped to damp out strand vibrations caused by vibration of the last 20 beams, thus preventing injury to the Type I specimens which were starting to set.

Since it was initially desired to study the shrinkage and creep in the loaded concrete, an attempt was made to place, in the beams of Series I, brass measuring plugs on a 10-in. gauge length with the intent of using a 10-in. Whittemore strain gauge to determine the changes in gauge length. Difficulty was encountered with inserting the brass plugs into the fast-setting, low slump concrete. The plugs could not be inserted deeply enough in the beam before striking large aggregate. For this, and other reasons, it was decided to install the measuring plugs prior to testing. This eliminated the opportunity for shrinkage studies between Type I and III cements, but did provide for creep investigations while the specimens were undergoing tests.

Curing and Storage

After all specimens were poured and the cylinders placed along the interior sides of the curing bed, two and three layers of moistened burlap were placed over the specimens in the steam and moist compartments, respectively (Fig. 17, Appendix). When the concrete had attained an initial set after approximately three hours, the steam was injected into the steam compartment for a period of 13 hours. The steam was transferred from the boiler to the bed by a hose and pipe system with the pipes placed between the lines of molds. A tarpaulin was draped and anchored over this compartment during the steam-curing period. In all cases, soaker hoses were placed on the burlap covering the specimens to insure keeping the burlap moist.

During the curing of Series I and Series V, an accurate 14-hour temperature record in the steam and moist compartments, and in certain beams of each compartment, was obtained by a continual recording temperature potentiometer. Under tentative plans, the temperature in the steam side would have been increased to 140° F at a rate of rise less than 40° F per hour. The actual temperature was somewhat lower than that planned. Thermocouples in the steam-cured beams revealed a temperature of approximately 125° F, while the temperature in the steam compartment was approximately 130° F in both recorded series. Moist-cured beams, in a compartment open to outside air, maintained a temperature of 75° F throughout the recorded period. The temperature in all series for the 14-hour interval should approximate those obtained in Series I and Series V, although the recording instrument was not available on the other days of pouring.

Upon termination of the steam-curing period, the cylinders were stripped of their fiber molds, and one moveable beam form in each line was loosened and parted from the beams to allow better curing. All specimens were allowed to finish the 3½-day curing period under moist conditions.

Before the specimens were removed from the bed, they were carefully identified in accordance with the notation shown in the appendix. Each specimen was referenced by this notation throughout the tests.

At the finish of the curing period, the concrete had far surpassed the 4000 psi strength specified to safely enable the transfer of force from strand concrete. Since a trial series of beams had been poured previously, to test the tensioning, molding, and beam removal technique, it was known that the strands could be cut with an acetylene torch without producing undesirable effects on the beams of small cross section. After removal, the strands were burned flush with the ¼-in. thick plate bulkheads, thus enabling the plate to be readily removed for re-use. Heat transferred from the steel plate to the concrete was thought to have no detrimental effect on the specimen.

Two cylinders representing each of the cement types and method of curing in each

series, or a total of eight cylinders per series, were submerged in water maintained at 70° F to obtain their 28-day potential. The remainder of the specimens were prepared for shipment, and stored at room temperature and humidity.

Essentially the same preparation procedure was followed throughout the five series which were molded as rapidly as posible. Considering that it required two days to remove the specimens and prepare the bed for the next pour, plus a 3½-day curing interval, series pours were made at approximately one week intervals.

TESTING PROCEDURE

Upon completion of Series V, all beam specimens were transferred to Kansas State University, Manhattan, Kansas, to undergo rigorous durability testing.

Inasmuch as the gauge plugs inserted in Series I beams did not provide sufficient accuracy to measure the change in gauge length for creep studies during the tests, plugs fashioned 1½-in. in length from ⅜-in. diameter brass stock were installed with a neat cement in the Series II specimens.

After the preparatory work and initial test readings were recorded, Series II specimens were placed in the freeze and thaw apparatus (Plates IV and V) to undergo a minimum of 400 repetitive fast cycles. These tests were conducted as nearly as practicable in compliance with ASTM Specification C291-527 for the "Tentative Method of Tests for Resistance of Concrete Specimens to Rapid Freezing in Air and Thawing in Water." The freeze and thaw method of testing durability was intended to determine the effects of variation in the properties of concrete on the resistance of the concrete to the freezing and thawing cycle specified in the method.

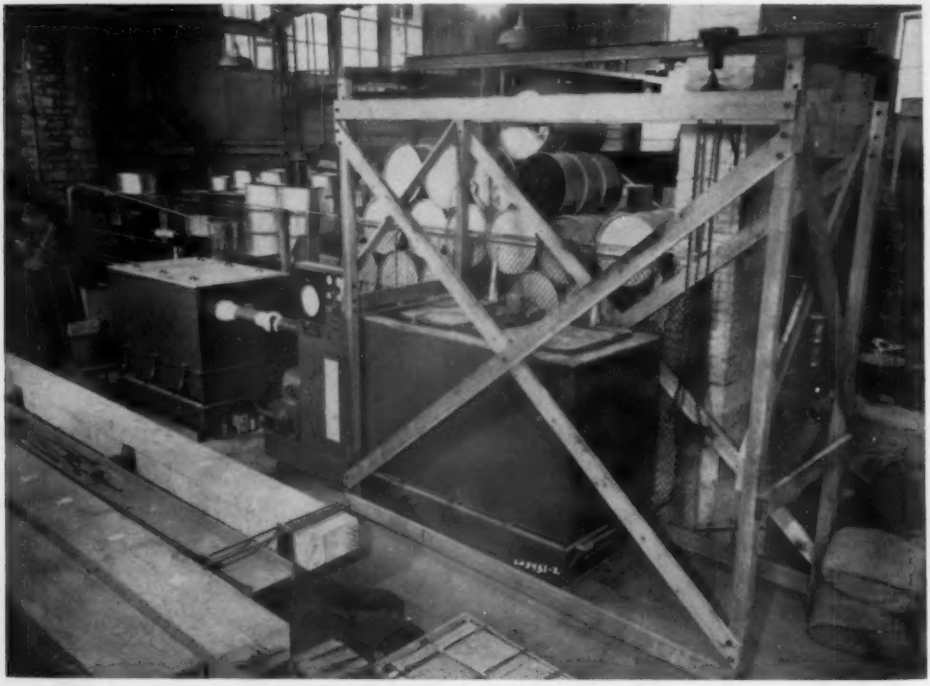


PLATE IV

Freezing and thawing apparatus used in tests.

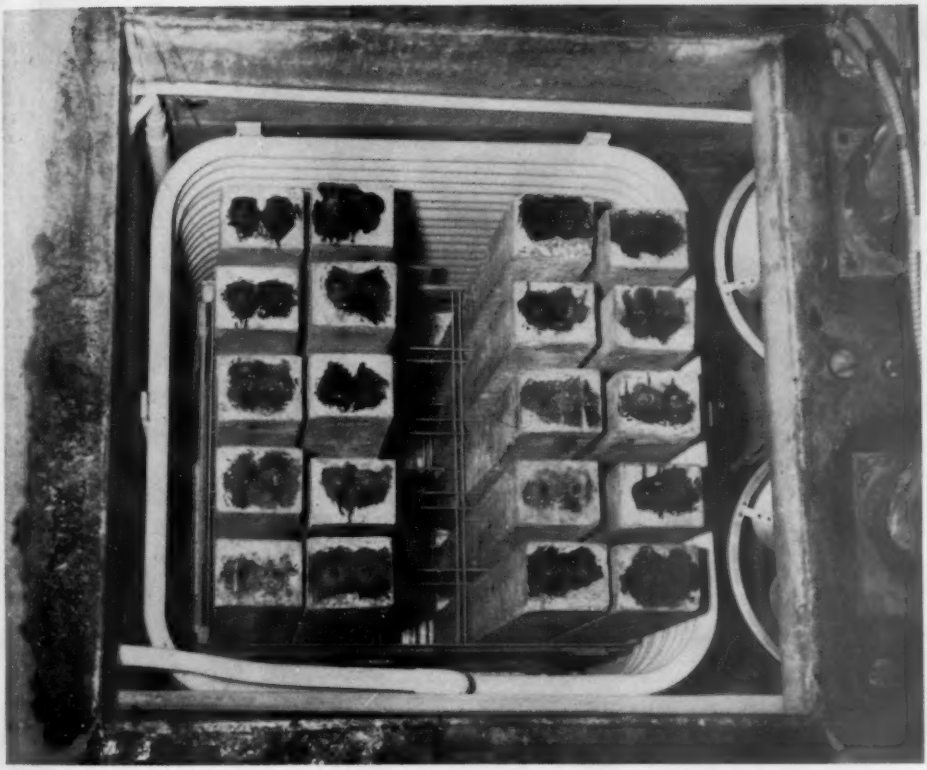


PLATE V

Subseries of beams shown in position within one of the two freezing and thawing compartments of the machine.

During the freezing period of the cycle, the temperatures in the beams were reduced from 40° F ± 3° F in a 3-hour period of time. The temperature at the center of the specimens during the thawing phase of the cycle was raised from 0° F ± 3° F to 40° F ± 3° F in a time of one hour. This made possible six complete cycles every 24 hour period.

The proper cyclic period was established by use of temperature potentiometer readings taken from a thermocouple installed in the center of a beam specimen. Temperatures in the freezing compartments dropped to −3° F in 1½ hr. and were maintained at this level for another 1½ hr. which was a sufficient period to permit the specimens to attain the proper temperature. The thaw water, which was automatically pumped from the water reservoir to the freezing compartment at the end of the freeze cycle and back to the reservoir at the termination of the thaw cycle, was maintained at a temperature of between 40° F and 50° F.

Specimens were removed from the apparatus at intervals of 50 cycles and transferred to the testing laboratory, (Plate VI). Determination of dynamic modulus of elasticity according to ASTM Specification C215-55T for "Fundamental Transverse Fequencies of Concrete Specimens," weight change, and change in gauge length measured with a 10-in. Whittemore gauge, aided analysis of the effect the freezing and thawing had on the durability of the specimens.

In order to alleviate the continual change in weight of the specimens due to absorption

of water during the thaw cycle, the beams were first read dry, and then submerged in water for approximately two weeks whereupon readings were again taken. Weight difference between dry and wet beams averaged approximately 0.5 lb. An increase in length was prevalent also.The saturated specimens were then placed in the baskets, lowered into the freeze and thaw compartments, and subjected from 2 to 10 cycles, after which the beams were removed and

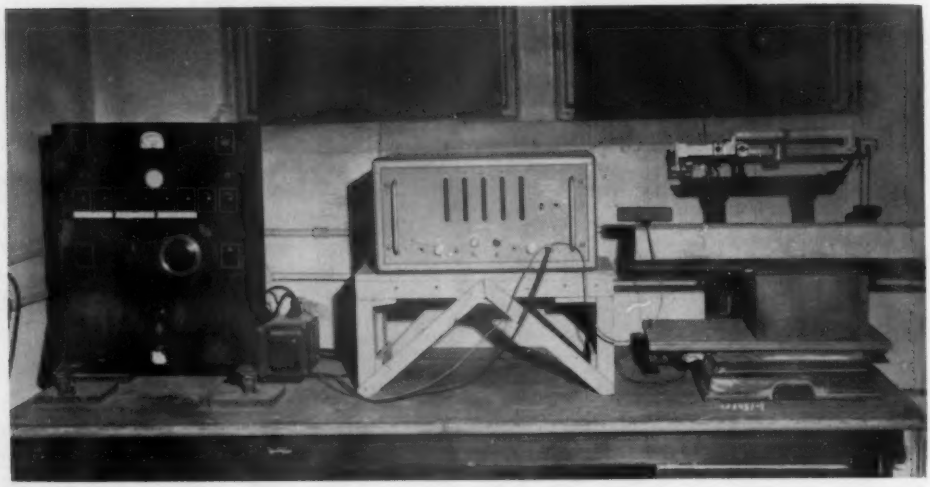


PLATE VI

Apparatus used in testing for durability of specimens.
From left to right: Oscilloscope, Frequency Meter, Scales and Driver.
Beam is shown in position for testing.

tested. The readings thus recorded were taken as the initial values and all succeeding readings were referenced to these. An individual durability record was kept for each beam specimen tested.

During subsequent sonic, weight, and gauge-length testing occasions, a basket containing a subseries of specimens at a temperature of 40° F was removed from the apparatus and transferred to a laboratory maintained at approximately 70° F. Since each beam was read individually, the subseries of specimens in each basket would slowly begin to warm. The specimens were, therefore, read in the same order as on the day when their initial readings were obtained. This procedure was designed to minimize any error, due to temperature differential, in gauge readings relative to preceding measurements. This error would arise if the beams were not tested in an orderly fashion as a consequence of the expansion of the specimens with increased temperature.

Series II was followed by Series III after the former specimens had been subjected to 400 cycles. Measuring plugs were installed in the beams of Series III and the tests were conducted in much the same manner as in Series II.

After Series III specimens had attained 400 cycles of exposure, a revised series composed of certain stress levels of Series II and Series III beams underwent 400 additional cycles. This gave the beams shown in Table 3 a total of 800 fast freezing and thawing cycles.

VOLUME TIGHTLY BOUND

Table 3. Revised series of specimens subjected to a total of 800 cycles.

Series	Stress Level
II	0
	2000
III	0
	1000
	2000

The tests have been discontinued for the present; however, the beams are available for further investigation, if this seems desirable.

EXPERIMENTAL RESULTS

Age-Strength Relationships

Analysis of how the variables of type of cement and method of curing affected the compressive strength of the concrete is given in Figs. 1 through 12. Compressive strength of the concrete was plotted againsts age in days from the data available in Table 7 of the appendix. Results for Series II and Series III are presented separately, with consideration being given to the combined compressive strength tests of all five series.

Series II. Examination of Figs. 1 and 2 for the first series tested indicates the existence of a strength differential between the concretes made with Type I and III cements in favor of the latter, as would be expected from tests at early ages. Steam curing resulted in somewhat increased strength at early ages, but otherwise exhibited no great advantage over the moist-cured specimens, (Figs. 3 and 4). Both batches of concrete of Series II had an average air content of approximately 3.4 per cent, and a slump measurement of ¾-in.

Series III. The results for Series III shown in Figs. 5 through 8 are somewhat different from those of Series II. There seems to be no essential strength difference between concretes made with Types I and III cements. These results can be partially explained upon observation of the air contents and slump measurements recorded for Series III during the molding operations. The first batch showed an air content of four per cent, with an average slump of ½-in. Readings for the second batch containing Type III cement were 4.8 per cent air and 2¼-in. slump. The loss of strength due to the combination of increased air content and water-cement ratio in the second batch, accounts largely for the erroneous impression given by Figs. 5 and 6 that there is no early age strength advantages to be offered by use of Type III rather than Type I cement.

Steam curing resulted in an increased strength at early ages, as it did in Series II; although Figs. 6 and 8 indicate no advantage over the moist-cured specimens as the age increased.

Combined Series. Figures 9 through 12 show a graphical representation of the combined compressive strength data for all five series. The resulting Figs. 9 through 10a again show the existence of an early age strength differential in favor of the concrete specimens representing Type III cement. Steam curing produced a slight strength increase during early curing stages but otherwise showed no advantage over the moist cured concrete, (Figs. 11 and 12).

Generally, 400 psi strength concrete was obtained by the second day of curing, although the specimens which were prepared from Type I cement and moist cured, required three days to develop such a strength. In three of the five series poured, those specimens containing Type III cement which were steam cured developed sufficient strength within one day after molding to safely enable the force in the steel strand to be transferred to the concrete.

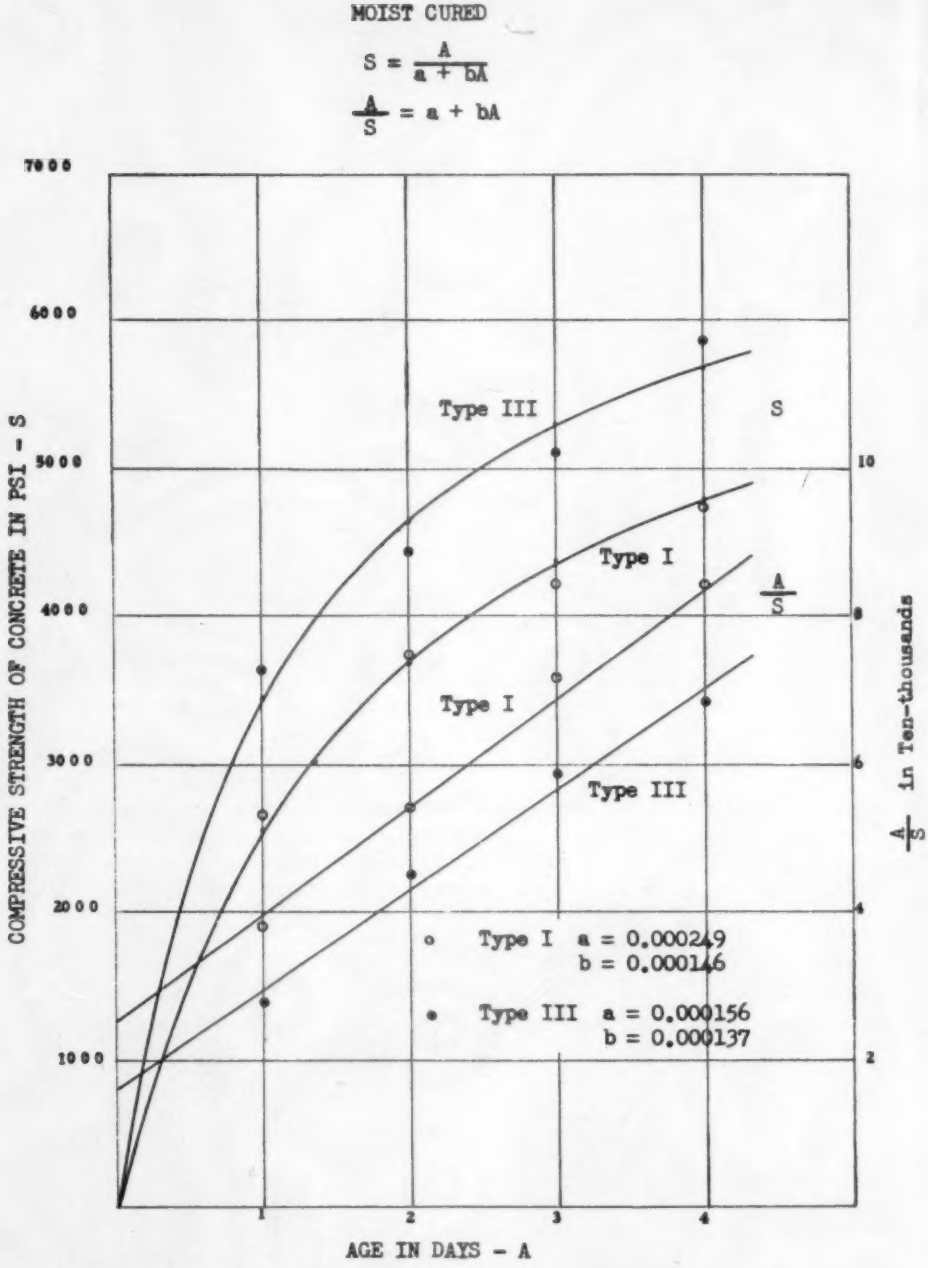


Fig. 1. Age-strength relationships for Series II specimens.

STEAM CURED

$$S = \frac{A}{a + bA}$$

$$\frac{A}{S} = a + bA$$

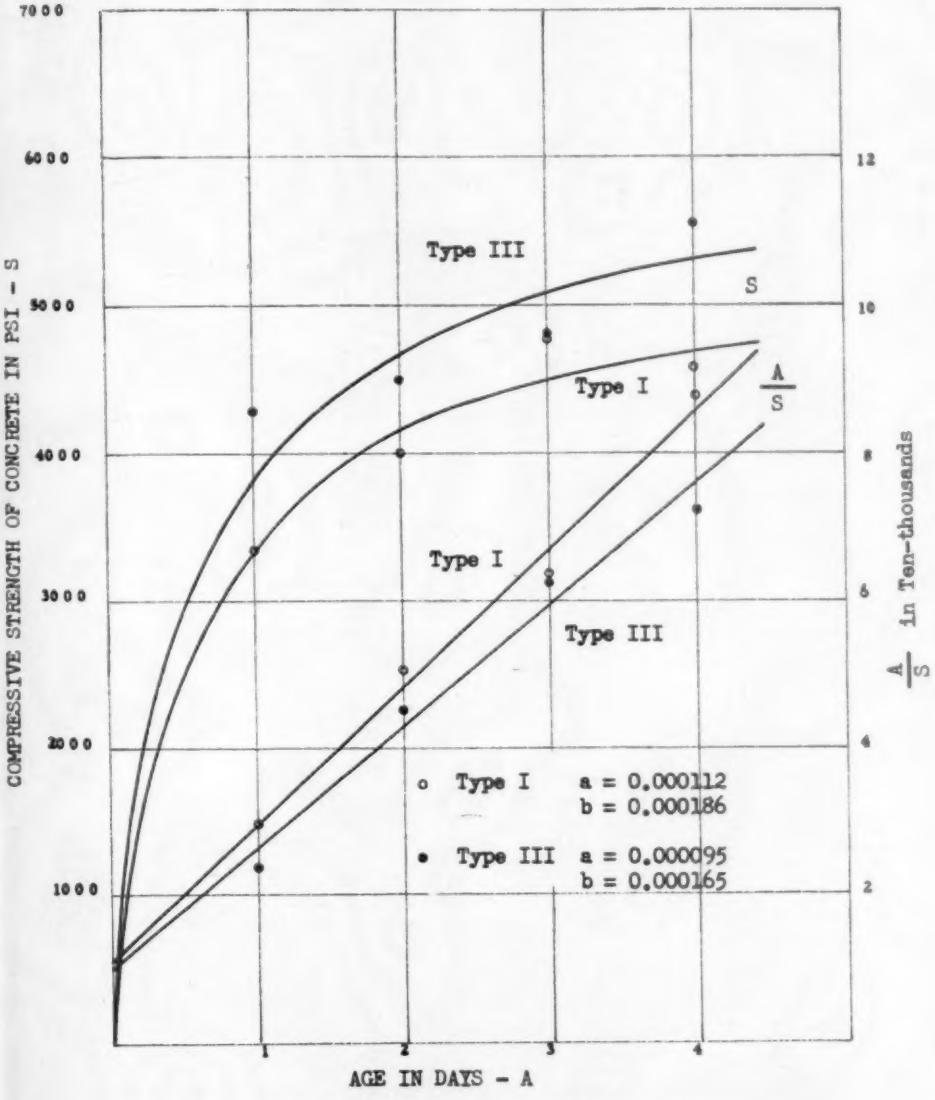


Fig. 2. Age-strength relationships for Series II specimens.

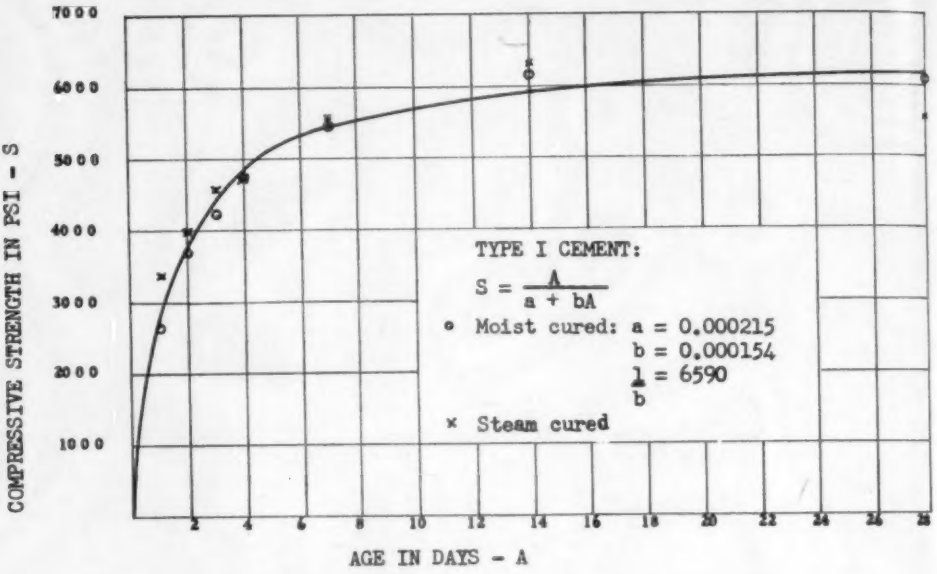


Fig. 3. Age-strength relationships for Series II specimens.

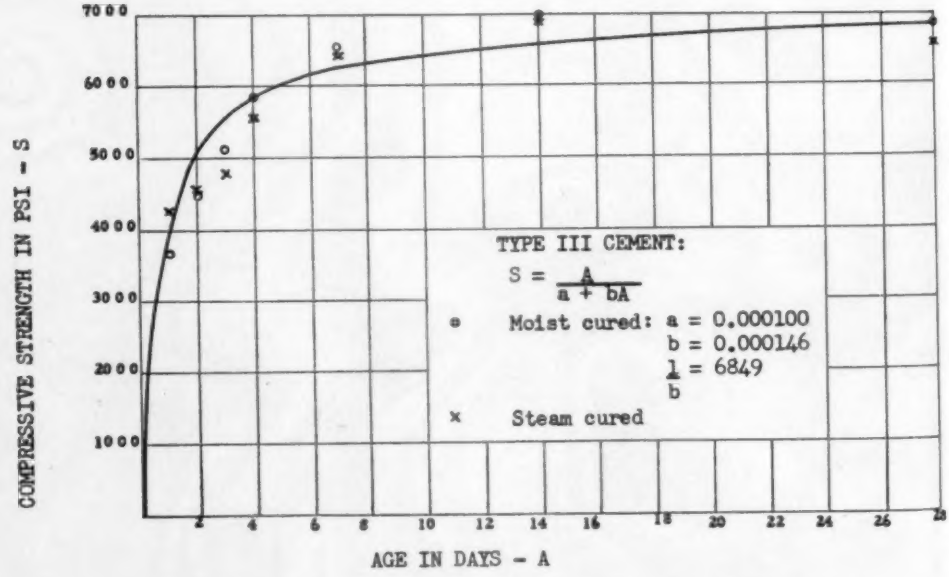


Fig. 4. Age-strength relationships for Series II specimens.

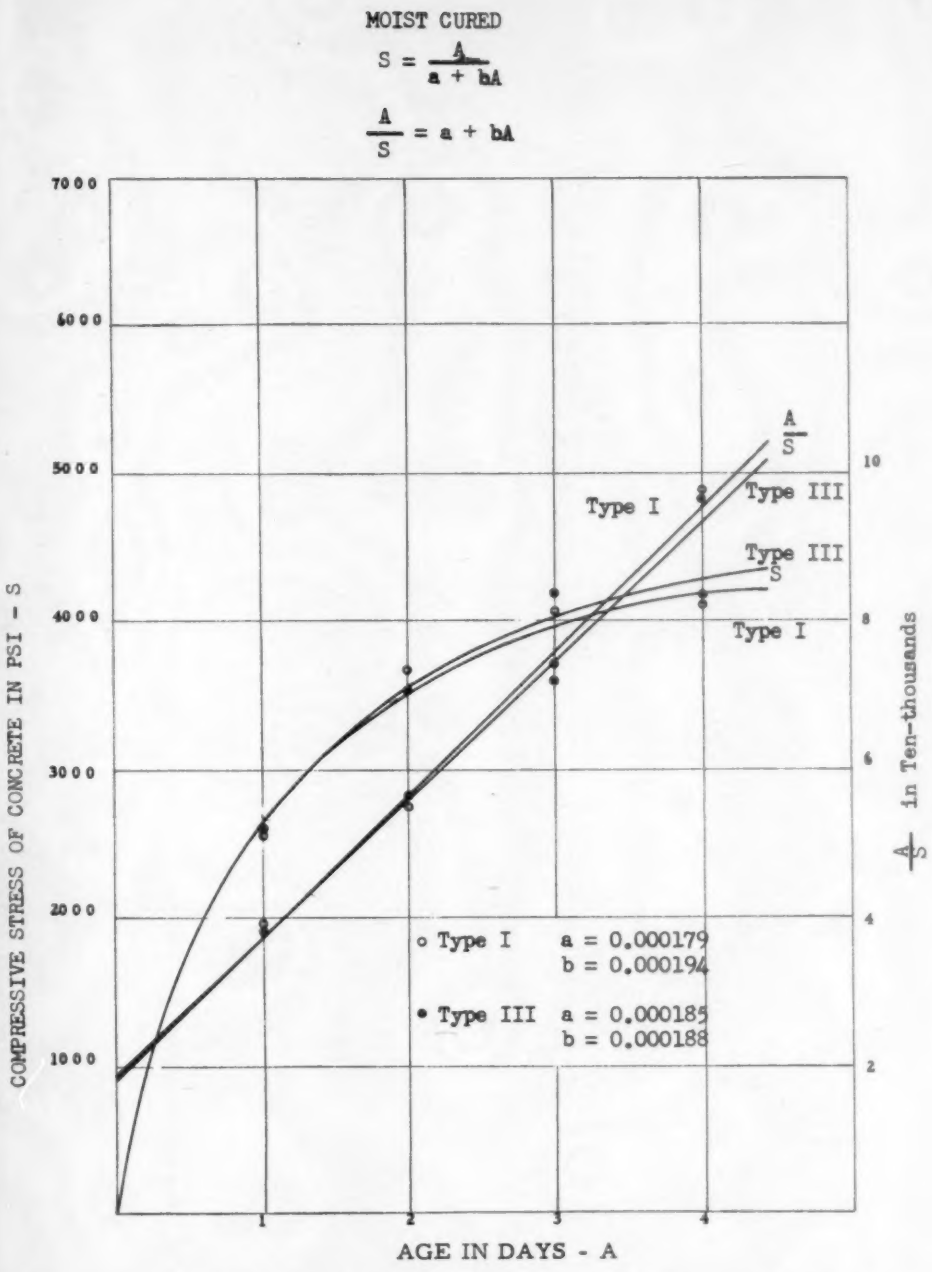


Fig. 5. Age-strength relationships for Series III specimens.

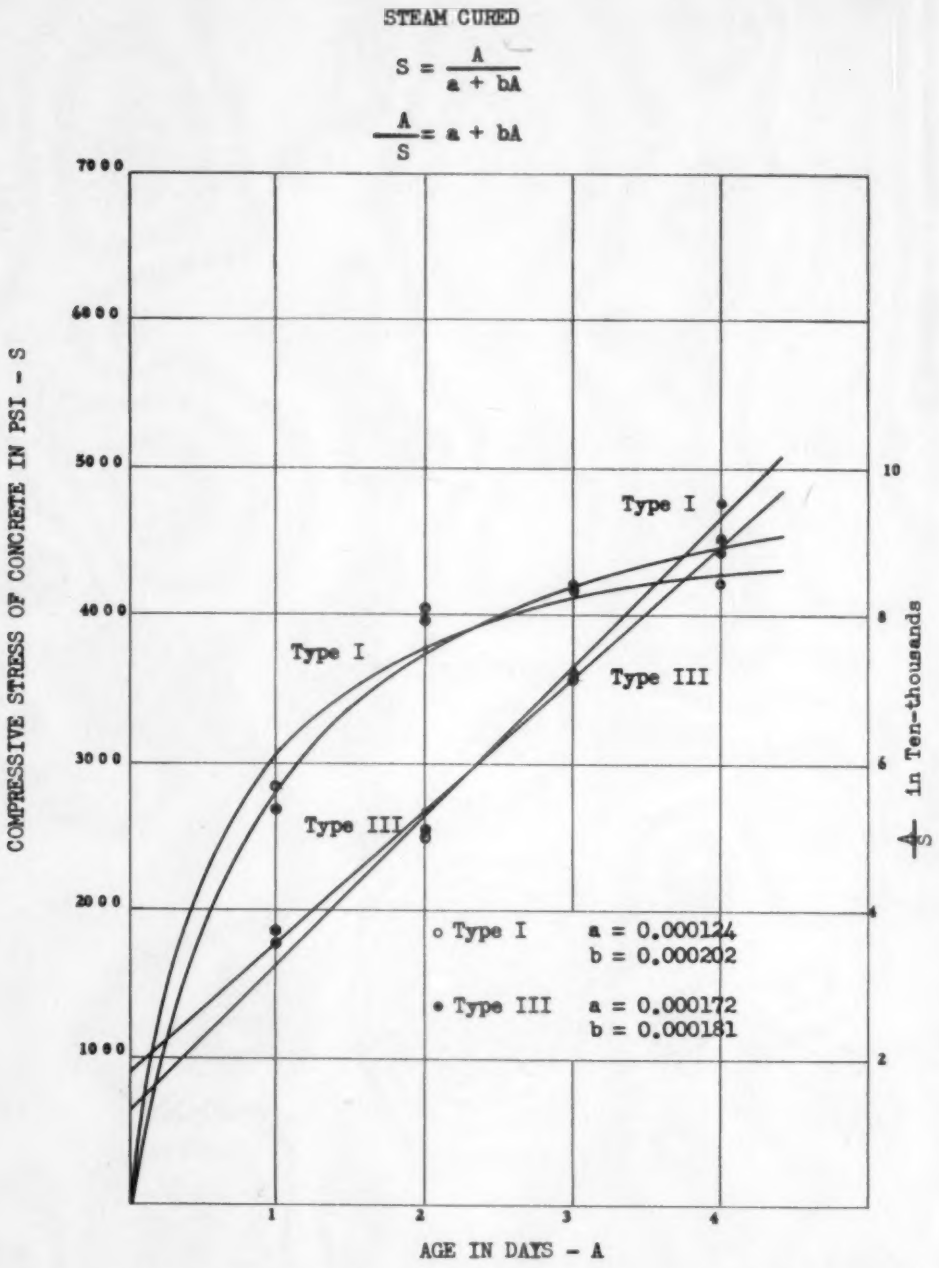


Fig. 6. Age-strength relationships for Series III specimens.

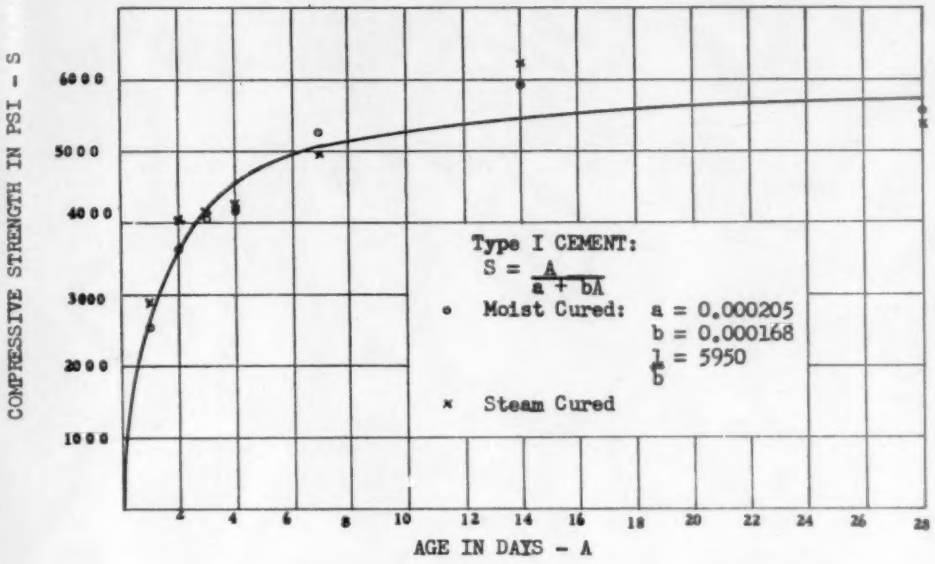


Fig. 7. Age-strength relationships for Series III specimens.

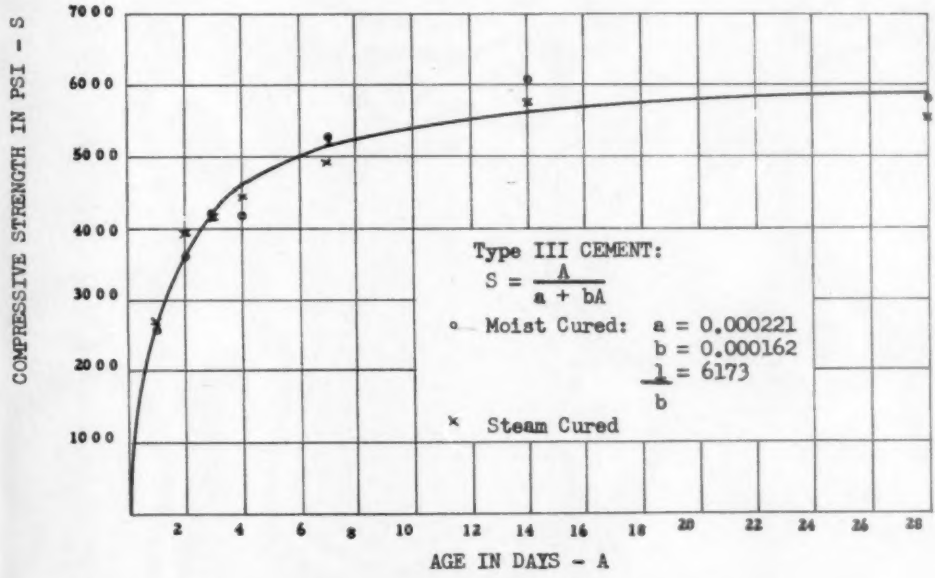
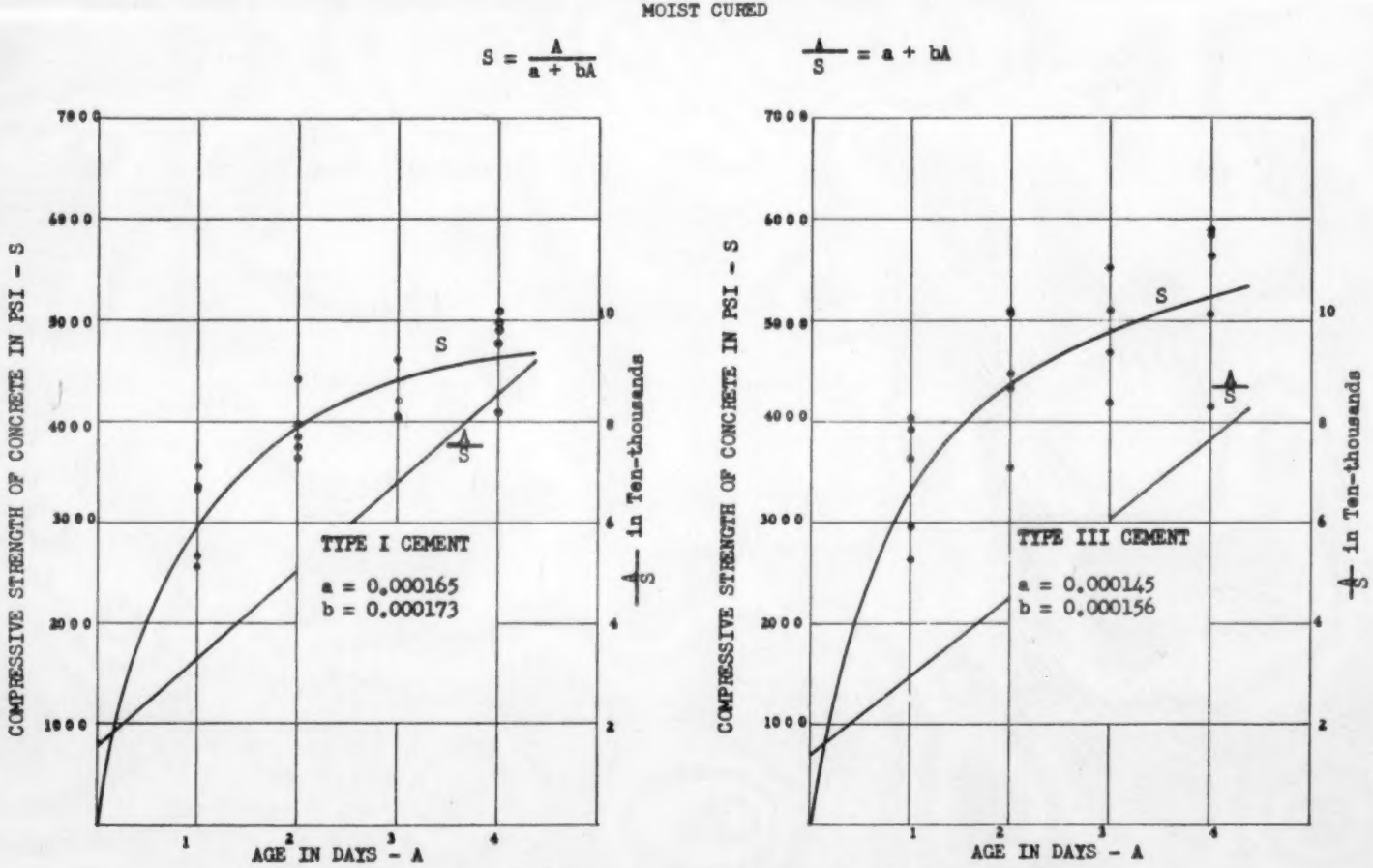
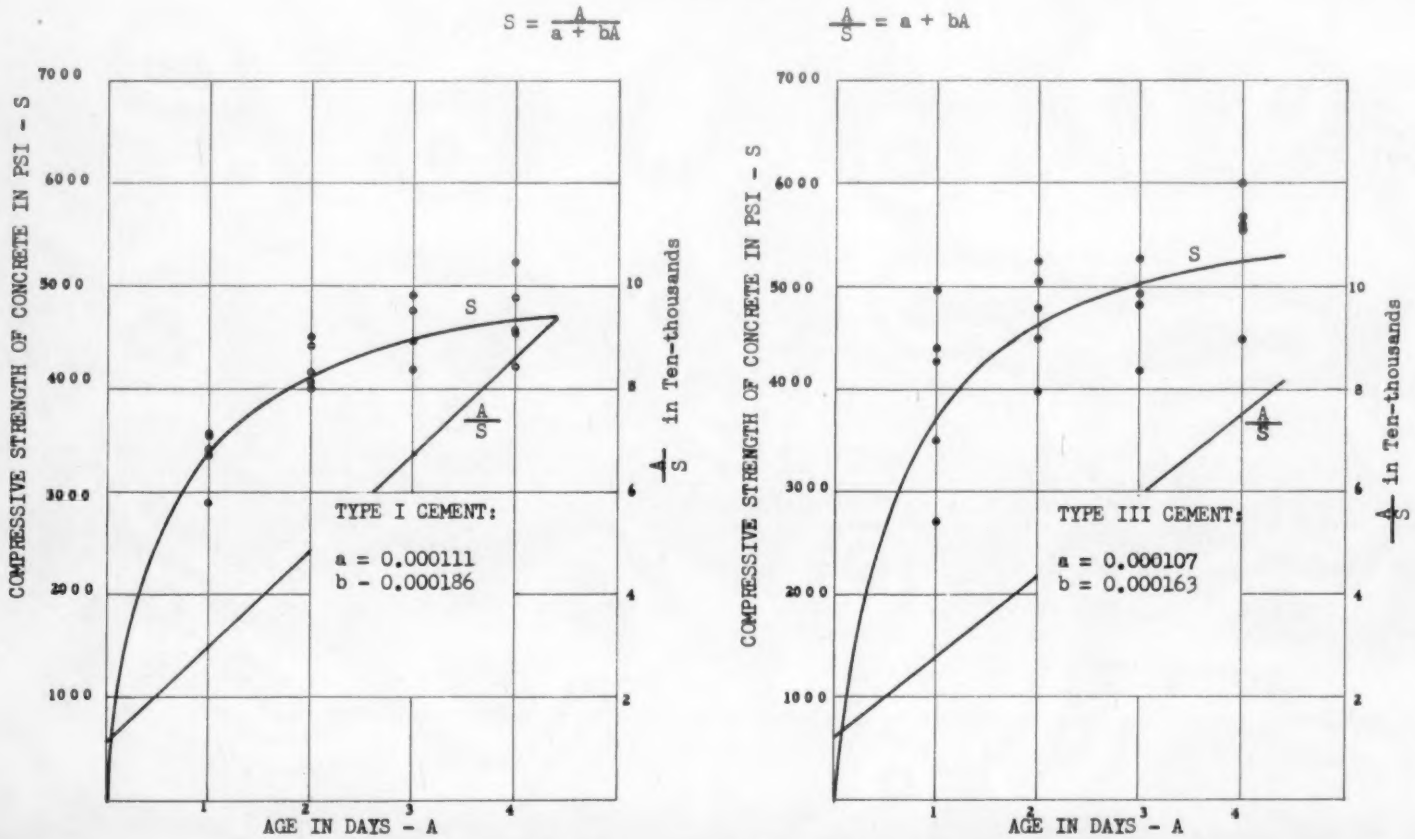


Fig. 8. Age-strength relationships for Series III specimens.



Figs. 9 and 9a. Age-strength relationships for all series combined.

STEAM CURED



Figs. 10 and 10a. Age-strength relationships for all series combined.

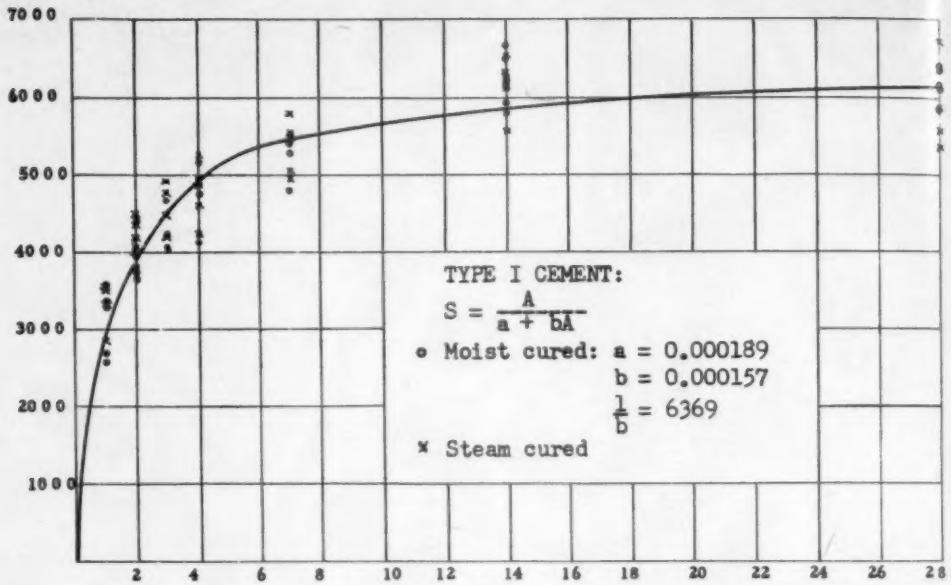


Fig. 11. Age-strength relationships for all series combined.

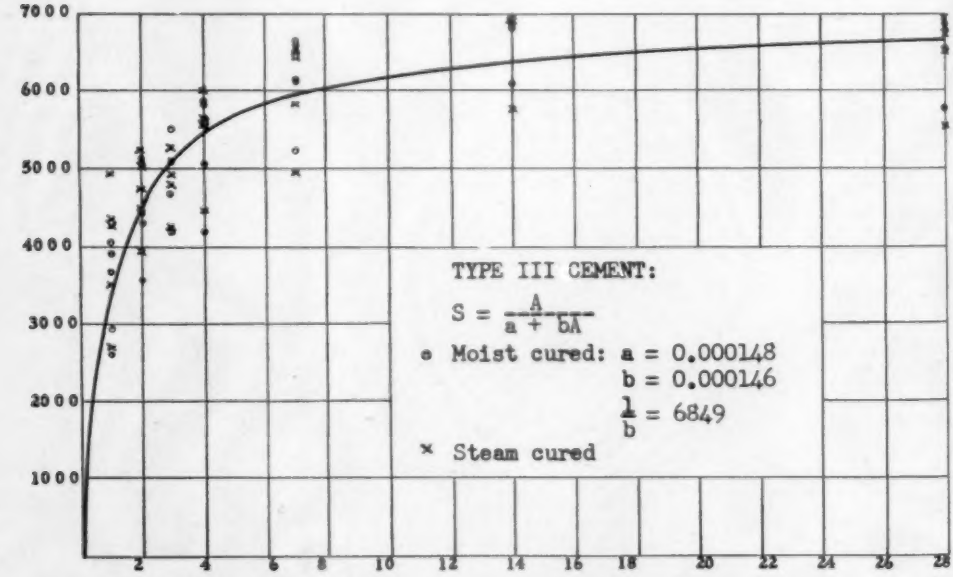


Fig. 12. Age-strength relationships for all series combined.

Durability of Prestressed Concrete in Freezing and Thawing

As mentioned previously, the specimens used in the freeezing and thawing tests were prestressed concrete prisms with a 3- by 4-in. cross section, and 32-in. length. The 40 beams

in each series tested were subjected to 400 repetitive fast cycles. The dynamic modulus of elasticity were determined periodically to observe the degree of reduction from the initial modulus which would give an indication of the deteriorating effect of the freezing and thawing. Time permitted only the testing of specimens from Series II and Series III. Results of these tests are recorded in Tables 8 through 10 in the appendix. Generally, readings were taken after every 50 cycles. However, since no large variation from the initial modulus or initial gauge length occurred, the observed values are reported only every 100 cycles to facilitate condensation of the tables.

The dynamic modulus of elasticity was calculated as follows: $E = CWn^2$ where E is the dynamic modulus, C is a constant for any one specimen ($C = 0.7875$), W is the weight of the specimen in pounds, and n is the fundamental transverse frequency of the beam in cycles per second.

Series II. Tables 8 through 8c pertain to the observed test readings for specimens of Series II. Examination of this data reveals that the soundness of the specimens on the basis of the dynamic modulus of elasticity determination appear very satisfactory. After 400 cycles of freezing and thawing, the durability factor of all beams was approximately 100 per cent. Needless to say, the effects of the introduced variables (level of stress in the concrete, type of cement, and method of curing) on the durability of the prestressed concrete could not be differentiated, or indeed, even noticed. There was no apparent visual deterioration of any of the beam specimens after having been subjected to 400 cycles, and only a negligible loss in weight.

Midway through the tests, difficulty was encountered with the testing apparatus used in obtaining the fundamental frequency of the beam specimens. This accounts for the numerous blanks found under the dynamic modulus heading in Tables 8a through 8c. Rather than stop the freezing and thawing tests while the trouble was being corrected, it was decided to continue with the periodic readings during which weight and gauge lengths could still be recorded.

Series III. Information concerning dynamic modulus and weight of Series III specimens is given in Tables 9 through 9c. The results generally reflect those of Series II in that the soundness of the specimens, on the basis of their dynamic modulus, appear to be very satisfactory after 400 cycles. The weights of the specimens remained essentially constant throughout the 400 cycles of exposure.

The results thus far are closely in agreement with those obtained by the State of Illinois, Bureau of Materials. The Bureau conducted tests to study the early-age-strength development of concrete made with Type III cements. For these tests, a number of 3- by 4- by 15-in. beams were made containing crushed stone coarse aggregate, and having a cement content of 5.8 bags per cubic yard of concrete. These were subjected to 400 fast freezing and thawing cycles. The concrete had an entrained air content of approximately 4.2 per cent and a slump of 2.75 in. Of the nine different brands of cement tested, only specimens made from each of four different cements experienced any apparent deterioration on basis of dynamic modulus of elasticity readings taken after periods of repetitive freezing and thawing. Of these four, specimens containing two of the cements that were tested had a final modulus which was 96 per cent of the initial, while readings of specimens made from the other two cements were 99 per cent of the initial modulus. Beams containing any one of the remaining five Type III cements encountered no reduction in dynamic modulus after 400 cycles.

Only one brand of cement was used in this investigation with the cement content being maintained at 7.0 rather than 5.8 bags per cubic yard of concrete. Slumps were generally lower; however, the amount of air-entrainment approximated that used in the tests conducted by the Illinois Bureau of Materials.

Special Series. A special series composed of beams of various stress levels from Series II and III was subjected to 400 additional cycles. Examination of dynamic modulus values in Tables 10a through 10b reveals that the soundness of all specimens is quite satisfactory. The only visual deterioration noticeable after 800 cycles was a slight surface spalling of

the beams. This, however, was not excessive nor more prevalent for specimens containing Type III cement than for those representing Type I cement.

Prior to testing the special series, four 1- by 2- by 16-in. specimens were cut from two of the regular test beams of Series III. One beam contained Type I cement, and the other Type III. These smaller specimens were then subjected to the 400 additional cycles with no visible deterioration occurring.

It may be noticed that the beams of Series II, zero psi stress level, have a comparatively lower dynamic modulus for the 400 (dry) readings. Close examination of the beams revealed minute cracks perpendicular to the longitudinal axis of the specimen. The probable explanation or cause of the cracks is that the beams received a severe shock after they had undergone 100 freezing and thawing cycles. There was no apparent visual damage at the time nor did the dynamic modulus readings change abruptly. After Series II had been subjected to 400 cycles, the specimens were stored at room temperature and humidity. As the beams dried, the cracks began to open and were noticed only after the frequency readings of the dry specimens were taken. It will be noticed that the modulus returns to approximately the initial reading after the beams have been submerged in water, indicating that the cracks closed due to the expansion of the concrete. Additional freezing and thawing appears to have little effect on the modulus as recorded after the 400 (wet) reading. The cracks were not noted in the few prestressed beams which were subjected to the same shock.

Since the prestressing steel is in rather small quantities, corrosion is to be a serious consideration in the durability or permanence of prestressed concrete. Generally, the cover problem is not so critical as in reinforced concrete because even if over-loading occurs, any resulting cracks in the section will close upon removal of the load.

Upon stripping the steel from a number of beams that had undergone 800 freezing and thawing cycles, it was found that not even the slightest hint of corrosion existed.

Examination of prestressed structures in France and England also indicate that prestressed concrete can be used with every confidence, (1). In France, prestressed transmission line poles designed by Freyssinet more than 30 years ago were found completely free of corrosion troubles. Prestressed beams made in England during 1940 were also examined but indications are that with the use of good quality materials, corrosion problems are not likely to occur.

Creep Investigations

Series II. Referring again to Tables 8a through 8c, it is noticed that throughout the tests of Series II there is but little variation from the initial gauge length of the specimens. Closer examination of the data compiled in Table 4 indicates that a number of trends may exist, however.

There appears to be a general increase in length of the specimens representing Type I cement, and also those specimens representing Type III that were subjected to little or no prestressing. However, the specimens containing high-early-strength cement that were subjected to a prestressing load of 1000 psi or greater, showed a decrease in length during the

Table 4. Index of change in length of Series II specimens (x 10^{-4} in.) (average of length changes observed at 50, 100, 150, 200, 250, 300, 350, and 400 cycles.)

Stress	Moist Cured		Steam Cured	
	Type I	Type III	Type I	Type III
0	+3.00	+4.29	+4.29	+3.29
500	+5.29	+1.14	+4.15	+2.72
1000	+0.14	−2.00	+1.71	−3.29
1500	+0.57	−1.29	−0.43	−2.29
2000	+0.86	−1.00	+2.14	−0.28

tests. Since the prestressing force tends to cause creep while freezing and thawing tends to cause expansion of concrete, one authority interpreted the observed decreases in length as an indication of greater creep in the concrete made with Type III cement than in the concrete

made with Type I cement. While agreeing that this conclusion may have some merit, another authority pointed out that on the information available, it could be just as easily concluded that the Type III cement concrete was of a superior quality that showed very little expansion in the freezing and thawing tests, and consequently an apparently greater contraction because of creep.

Series III. The results of the creep investigations representing Series III are given in Tables 9a through 9c. No very definite trend is observable in the changes of length of the specimens during the freezing and thawing tests (Table 5).

Table 5. Index of change in length of Series III specimens (x 10^{-4} in.) (average length changes observed at 50, 100, 150, 200, 250, 300, 350, and 400 cycles.)

Stress	Moist Cured		Steam Cured	
	Type I	Type III	Type I	Type III
0	+4.29	+1.67	+0.25	+1.12
500	+3.86	+0.33	+1.17	+3.62
1000	+7.00	+11.78	−0.71	+8.60
1500	−0.14	+0.20	+1.38	−3.38
2000	+1.00	+3.88	+4.17	−0.62

Deterioration of the neat cement used in fastening the gauge plugs in the specimens of Series III resulted in the loosening of a number of the plugs after approximately 150 cycles of freezing and thawing. Whenever a loose plug was discovered during measurement of the gauge length, the specimen was removed from the tests for 50 cycles at which time the plugs were reinserted. The beam was then introduced back into the tests at the time of the next periodical reading. An account was kept of the cycles each beam missed, which was generally only 50, and the tests were continued until all beams had been subjected to 400 cycles.

Since many of the gauge length readings had a tendency to be rather erratic, and evidently did not represent the true action of the concrete, they were omitted from the tabulations of changes of gauge length given in Tables 9a through 9c. For this reason, plus the increased chances for experimental error, the creep data obtained for Series III should not be regarded as having much significance in determining the effect of the two types of cement on creep action of the concrete.

It was believed that cement used in inserting the measuring plugs in Series III was partially hydrated or otherwise defective. The troubles previously described were not experienced with the specimens of Series II nor with those of Series III after the plugs had been reset.

Special Series. Table 6 is obtained from Tables 10a through 10b, which contain the experimental results for the special series comprised of beams of both Series II and III.

Table 6. Index of change in length of special series specimens (x 10^{-4} in.) (average of length changes observed at 450, 500, 550, 600, 700, and 800 cycles.)

Stress	Series	Moist Cured		Steam Cured	
		Type I	Type III	Type I	Type III
0	II	—	+3.00	+3.00	+3.50
0	III	+2.00	+5.40	+3.34	+0.00
1000	III	+2.00	+0.17	−0.17	−1.80
2000	II	+1.50	+1.33	+1.17	+2.00
2000	III	+1.17	+2.33	+1.50	+0.17

Even after being subjected to 800 cycles there appears to be but small changes in any of the specimens 10-in. gauge length. In fact, it is quite conceivable that experimental errors made in reading the gauge length could account for most of the variances, considering that even 5.4 x 10^{-4} in. is not much length change over a 10-in. gauge length.

CONCLUSIONS

The foregoing presentation and discussion of test results gives an indication of the advantages to be gained by the use of high-early-strength cement and steam curing. Sufficient strength will usually be generated in one day to allow the concrete to be removed from the prestressing bed. The concrete specimens using Type I cement required two, and sometimes three days to develop a 4000 psi compressive strength. Specimens representing Type III cement generally tended to give higher compressive strengths than those specimens containing Type I cement, even after 28 days had elapsed.

The soundness of all beam specimens after being subjected to 400 and as many as 800 fast freezing and thawing cycles was very satisfactory. The effects of the introduced variables (level of stress in the concrete, type of cement, and method of curing) on the durability of the concrete could not be differentiated or, indeed, even noticed. In no instance was the final dynamic modulus less than 97 per cent of the initial modulus. Slight scaling of a number of the beams after 800 cycles was the only visible deterioration noticed due to alternate freezing and thawing of the specimens. The weight loss due to this scaling was nominal.

Creep investigations conducted on the prestressed concrete beams undergoing the freezing and thawing tests did not result in any significant or consistent information regarding the effect of Type I as compared with Type III cement on the creep of the loaded concrete.

In Series II, a slight tendency for the beams representing Type III cement that were subjected to 1000 psi stress or greater to contract during exposure to freezing and thawing, while the remaining specimens tended to expand, brought forth the speculations that perhaps, since freezing and thawing causes expansion and compressive forces tend to cause creep in the concrete, there was greater creep in the concrete made with Type III cement than in concrete made with Type I cement. However, results from neither Series III or the special series that was subjected to 800 cycles showed indications of verifying this trend.

Actually, the observed changes in gauge length with respect to the initial gauge readings were quite small for all beams tested. Much of the variation can possibly be attributed to errors in testing which will certainly be present in investigations of this nature.

The strength, mixture design, and handling of the high quality concrete used in this investigation was very similar to that used in most actual prestressed concrete construction practice. It could be concluded, on the basis of these tests, that such concrete, particularly when under compression, is a highly durable construction material from the standpoint of weathering action. The proper use of Type III cement, as a means in obtaining high strength concrete at an early age, will not be detrimental to the durability of the concrete.

ACKNOWLEDGMENTS

For their guidance and cooperation, the writer wishes to express his appreciation to Mr. J. O. Whitlock, President of the Midwest Prestressed Concrete Company of Springfield, Illinois, for his helpful reviewing of the material presented in this thesis and also to Professor C. H. Scholer and Professor M. E. Raville, both of the Department of Applied Mechanics, Kansas State University, for their helpful comments and review of the thesis. The author takes this opportunity to thank also the State of Illinois, Division of Highways, Bureau of Materials, Springfield, Illinois, particularly Mr. O. Larson who analyzed much of the test results, and Mr. F. Blandin who was of great assistance throughout the tests. Thanks also to Mr. J. Butler, of Midwest Prestressed Concrete Company, who greatly aided in the preparation of the specimens, and to all who, in some manner, participated in the investigation.

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1. **Permanence of Prestressed Concrete**, Prestressing News, Prestressed Concrete Development Group of Groovenor Gardens, London, England, 1958.
2. **Tests of High-Early Strength Cements Available for Highway Construction in Illinois, State of Illinois,** Department of Public Works and Buildings, Division of Highways, Bureau of Materials, 1951. 18 p, Table 8(b).

APPENDIX

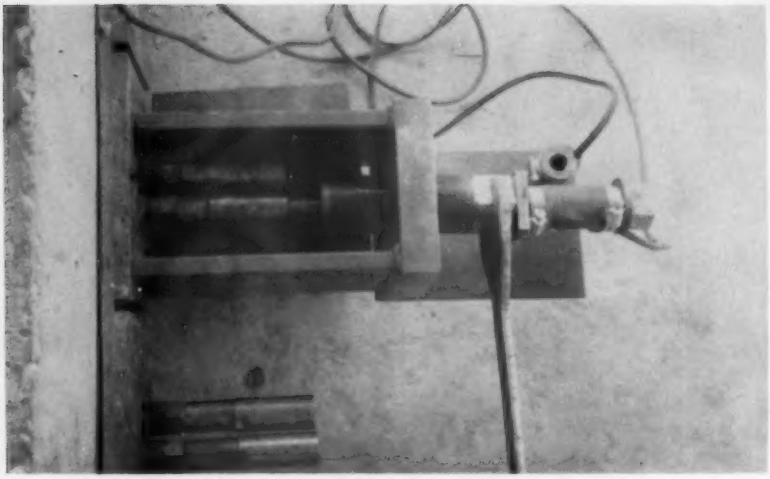
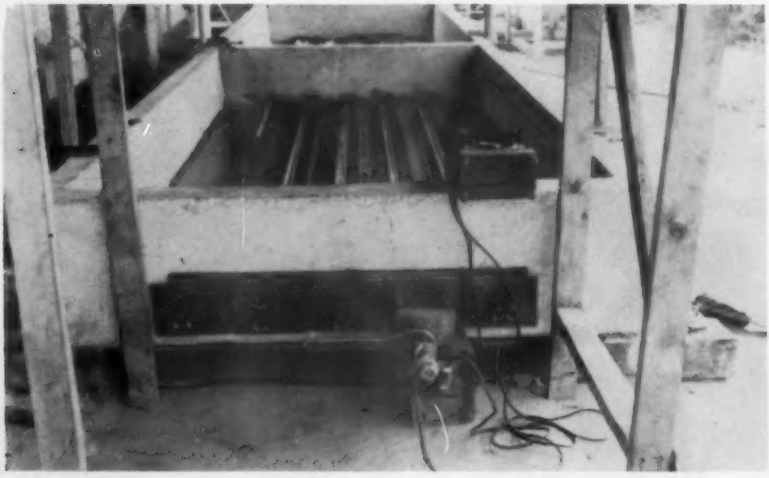


Fig. 13. Screw type jack used to pretension steel strand

Fig. 14. View of tensioning bulkhead along with jack and Baldwin SR-4 strain indicator



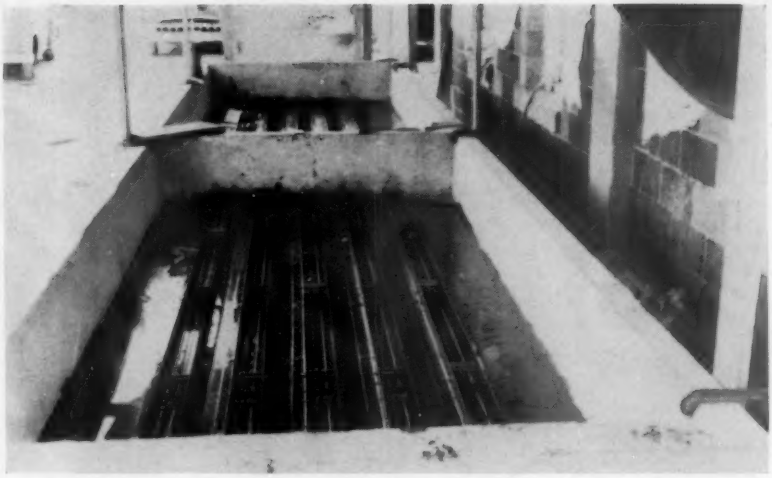
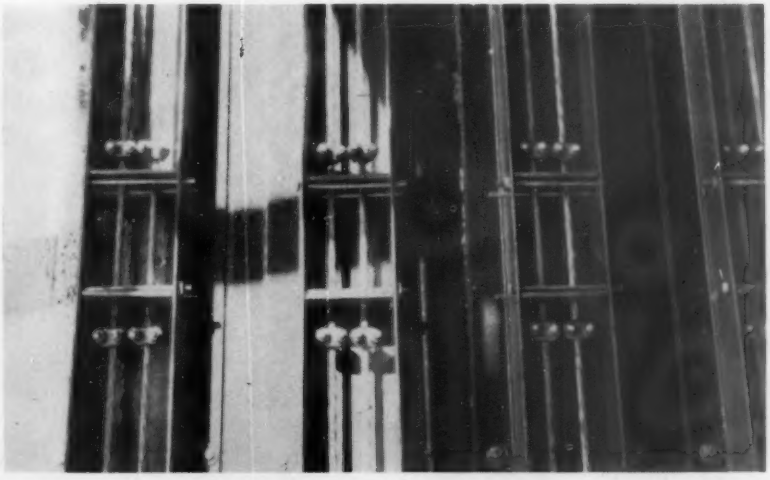


Fig. 15. View of bed just prior to molding test specimens. Bed has a capacity of 40 beams and 72 cylinders.

Fig. 16. Detail of beam mold ends. Note bond increasers and method of securing plate bulkheads.



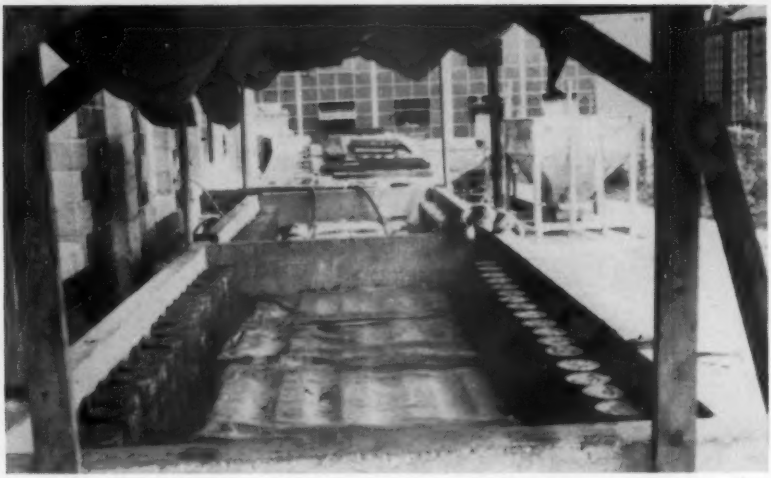


Fig. 17. View of bed after completion of molding one series of specimens. Moist burlap covered the beams and cylinders during the curing period.

Fig. 18. The cured specimens were carefully identified before removal from bed. Tarpaulin canopy kept bed and molds from becoming excessively warm during molding operations and early curing stages.

CONCRETE DESIGN USED IN TEST SPECIMENS

Crushed stone - Size B - Specific Gravity 2.69 Voids.
Sand - Specific Gravity, 2.63
Portland cement - Types I and III
7 bags per cu yd
3.86 cu ft per bag
4.0 per cent air
4.3 gal water per bag
0.79 mortar factor

Quantities per bag			Per ¾ cu yd
Cement	(lb)	94	494
Sand	(lb)	167	877
Stone	(lb)	275	1442
Water	(lb)	4.3	22.6
Abs. vol. cement			0.48 cu ft
Abs. vol. sand			1.02 cu ft
Abs. vol. stone			1.64 cu ft
Vol. water			0.57 cu ft
Vol. air			0.15 cu ft
Yield			3.86 cu ft

Note: Water given is for aggregate in saturated, surface dry condition. Sand, stone, and water to be corrected for actual field conditions.

Strand: **Three-eighths inch diameter steel strand (7 wires) manufactured by** the American Steel & Wire Co. of Waukegan, Illinois.
Spool C-302
Average breaking load of 5 samples composed of 2 strands each taken from the above spool - 21,123 lb.

Aggregates: **Crushed stone - Size B**
Producer - Lehigh Stone Co., Joliet, Illinois.
Laboratory No. 58-6251
Sampled from stockpile

Test Results

Specific gravity - dry	2.62
Specific gravity - surface dry	2.69
Absorption (per cent)	2.9
Abrasion loss (per cent)	3.33
Weight per cu ft (lb)	98.2
Voids (per cent)	41.5

Soundness - Na_2SO_4, 5 cycles
wt'd av. per cent loss - 12.98

Sieve Analysis

Sieve Size	Total Passing, %
1"	100
¾"	95
½"	75
⅜"	56
No. 4	10
8	4.1

Sand
Producers - Buckhart Sand & Gravel Co., Buckhart, Ill.
Laboratory No. 58-7595
Sampled from stockpile

Test Results

Specific gravity - dry	2.63
Specific gravity - surface dry	2.64
Soundness - Na_2SO_4, 5 cycles	
wt'd. av. per cent loss -	4.23

Sieve Analysis

Sieve Size	Total Passing, %
No. 4	100
8	97
16	72
30	48
50	27
100	9
200	1

Cement: **Standard Portland Cement (Type I) and High-early-strength Portland Cement (Type III)**
Manufactured by Medusa Portland Cement Co. of Dixon Ill.
Laboratory No. 58-6250

Test Results

	Type I	Type III
Autoclave expansion (%)	0.22	0.13
Time of set (Initial	3 hr 15 min	3 hr 15 min
Time of set (Final	5 hr 0 min	5 hr 0 min
Water for normal consistency (%)	24.0	27.0
Compressive strength (psi) (Standard mortar)		
24 hours		2442
3 days	2558	3917
7 days	3167	
Insoluble residue (%)	0.16	0.30
Ignition loss (%)	1.30	1.83
Sulphuric anhydride (SO_3) (%)	2.37	2.42
Magnesia, Mg O (%)	3.18	3.08

Note: All tests were made by the State of Illinois, Division of Highways, Bureau of Materials, Springfield, Illinois.

Quantities per bag of cement

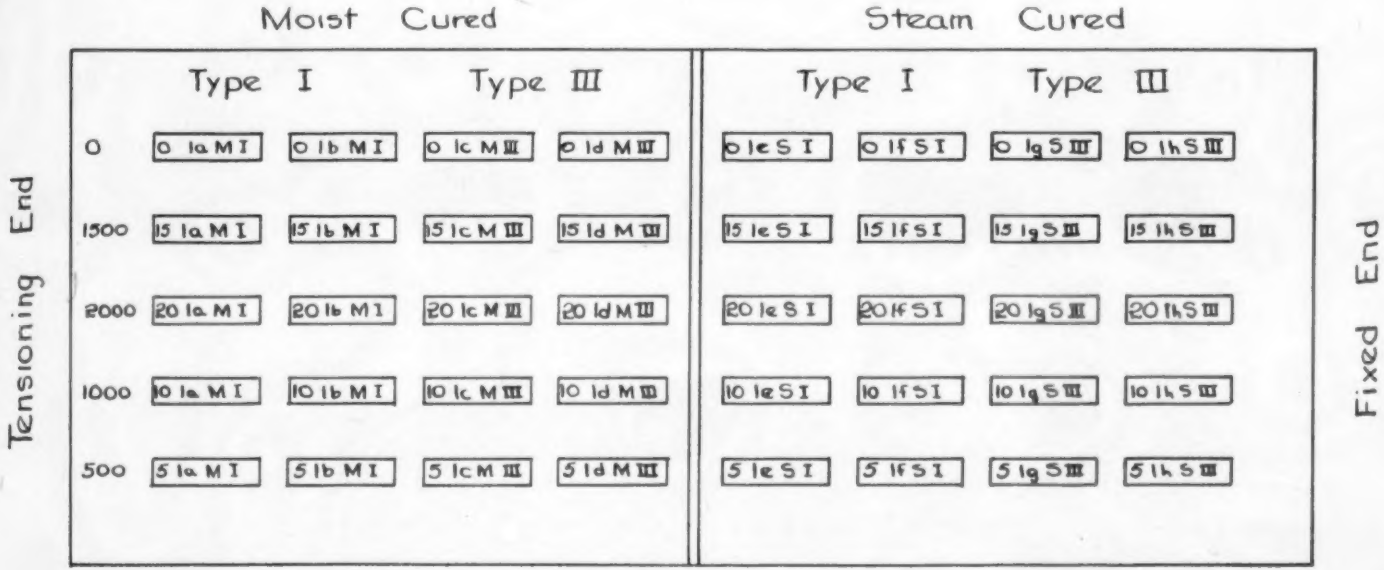
	Series	Type Cement	Sand lb	Stone lb	Water gal	Slump in.	Air Content %
Data on Concrete Mixtures	I	I	167	275	4.38	0.50	4.1
		III	167	275	4.75	0.75	3.7
	II	I	167	275	4.38	0.75	3.4
		III	167	275	4.57	1.00	3.3
	III	I	167	275	4.75	0.50	4.0
		III	167	275	4.80 ?	2.25	4.8
	IV	I	167	275	4.57	0.50	3.4
		III	167	275	4.40	0.50	4.4
	V	I	167	275	4.57	0.75	3.5
		III	167	275	4.68	1.00	3.6
						0.85	3.8

Table 7. Compressive strength of concrete determined from 6" by 12" cylinders (psi).

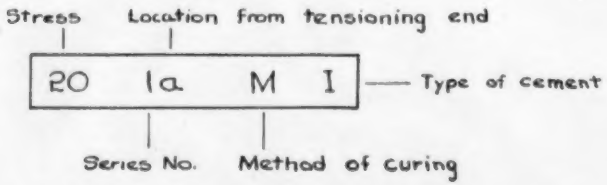
Age (days)	Series	Moist		Steam	
		Type I	Type III	Type I	Type III
1	I	3310	2920	3520	3500
	II	2650	3645	3345	4265
	III	2550	2600	2850	2690
	IV	3330	3910	3380	4370
	V	3555	4020	3540	4955
	Av.	3079	3419	3327	3964
2	I	3965	4300	4370	4775
	II	3715	4440	4000	4490
	III	3645	3560	4035	3965
	IV	3805	5115	4160	5010
	V	4390	5080	4480	5220
	Av.	3904	4499	4209	4692
3	I	4640	4690	4905	4920
	II	4210	5100	4745	4795
	III	4055	4190	4195	4180
	IV	4050	5505	4460	5225
	V	—	—	—	—
	Av.	4238	4871	4576	4780
4	I	4885	5060	5205	5650
	II	4760	5840	4570	5540
	III	4105	4160	4210	4425
	IV	4960	5645	4530	5560
	V	5150	5805	4865	6000
	Av.	4766	5302	4676	5455
7	I	5490	6160	5785	6460
	II	5450	6600	5525	6460
	III	5260	5235	4970	4905
	IV	4795	6145	5030	5860
	V	5415	6655	5490	6480
	Av.	5282	6159	5360	6033
14	I	6480	6885	6530	7135
	II	6160	7120	6300	6955
	III	5930	6070	6215	5730
	IV	6110	6815	5560	7080
	V	6670	6960	5735	7240
	Av.	6270	6779	6068	6828
28	I	6390	6850	6710	6920
	II	6110	6530	5560	6535
	III	5595	5790	5310	5525
	IV	6160	6765	5855	7425
	V	5855	6850	5840	6750
	Av.	6022	6557	5855	6631

Note: All values of compressive strengths for each series are the average of two cylinder breaks.

Plan of Bed Showing Arrangement of Specimens



Notation



Information Concerning the Forthcoming Tables

Series II

Date of molding — July 31, 1958.

Date that series was first subjected to freezing and thawing test — October 28, 1958.

Date that series was removed from test — December 30, 1958.

Series III

Date of molding — August 6, 1958.

Date that series was first subjected to freezing and thawing test— December 31, 1958.

Date that series was removed from test — March 12, 1959.

Special Series

This series was composed of beams from Series II and Series III.

Date of molding — given above.

Date that series was first subjected to freezing and thawing test — March 31, 1959.

Date that series was removed from test — June 10, 1959.

Table 8a. Dynamic modulus of elasticity, weight, and change in gauge length of 3" by 4" by 32" prestressed concrete beams after various periods of repetitive freezing and thawing (fast cycle) - Series II.

The first line in each tabulation under dynamic modulus shows initial modulus in millions of psi. All other values are in per cent of the initial modulus.

A (+) sign and (-) sign denote an increase and decrease, respectively, from the initial gauge length.

Zero psi Stress

MOIST CURED				STEAM CURED			
No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)	No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)
			TYPE I				
2	5.9×10^6	34.72	0.0	2	5.8×10^6	34.76	0.0
2	100	34.72	0.0	2	5.8	34.76	0.0
50	100	34.68	-3	50	100	34.72	-2
100	98	34.64	+2	100	100	34.70	+4
150	100	34.64	+2	150	100	36.68	+2
200		34.59	+7	200	100	34.66	+4
250	100	34.55	+7	250		34.56	+6
300				300		34.57	+10
350				400	102	34.58	+6
400							
			TYPE III				
2	6.1×10^6	35.25	0.0	2	6.2×10^6	35.65	0.0
2	100	35.27	0.0	2	100	35.65	0.0
50	100	35.24	-2	50	100	35.60	-2
100	98	35.20	+4	100	97	35.57	+2
150	100	35.20	+4	150	97	35.57	+2
200		35.17	4	200		35.55	+4
250		35.12	5	250		35.48	+6
300		35.10	9	300		35.48	+9
350				350			
400		35.15	6	400	97	35.51	+2

Table 8b. (contd.)

MOIST CURED				STEAM CURED			
No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length, (10^{-4} in.)	No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)
			TYPE I (500 psi stress)				
2	6.1×10^6	35.40	0.0	2	6.1×10^6	35.60	0.0
100	100	35.31	+6	100	100	35.55	+5
200		35.30	+4	200		35.56	+5
300		35.29	+1	300		35.56	+6
400	102	35.29	+17	400	103	35.54	+6
			TYPE III				
2	6.0×10^6	35.55	0.0	2	6.1×10^6	35.68	0.0
100	97	35.46	+2	100	100	35.61	+4
200		35.46	+1	200		35.61	+3
300		35.46	+2	300		35.60	+4
400	102	35.45	+4	400		35.57	+6
			TYPE I (1000 psi stress)				
2	6.0×10^6	35.83	0.0	2	6.2×10^6	35.68	0.0
100	98	35.78	+1	100	98	35.60	+4
200		35.78	+1	200		35.61	+2
300		35.78	+2	300		35.61	+2
400	105	35.78	-3	400	105	35.59	+4
			TYPE III				
2	6.2×10^6	35.69	0.0	2	6.1×10^6	35.60	0.0
100	100	35.66	+1	100	98	35.51	-1
200		35.62	-1	200		35.52	-4
300		35.62	-2	300		35.48	-2
400	103	35.62	-3	400	102	35.42	-4

Table 8c. (concl.)

MOIST CURED				STEAM CURED			
No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)	No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)
			TYPE I (1500 psi stress)				
2	6.1×10^6	35.49	0.0	2	6.1×10^6	35.42	0.0
100	98	35.40	+1	100	98	35.35	-1
200		35.40	+2	200		35.34	-1
300		35.39	+3	300		35.26	+4
400	103	35.41	0.0	400	103	35.26	-2
			TYPE III				
2	6.2×10^6	35.79	0.0	2	6.2×10^6	35.74	0.0
100	100	35.70	-2	100	97	35.68	-3
200		35.70	-2	200		35.65	-4
300		35.57	0.0	300		35.59	+1
400	103	35.64	+2	400	103	35.59	-5
			TYPE I (2000 psi stress)				
2	6.1×10^6	35.26	0.0	2	6.5×10^6	35.60	0.0
100	95	35.15	0.0	100	95	35.52	+4
200		35.16	+1	200		35.50	+1
300		35.11	+6	300		35.48	+6
400	102	35.10	0.0	400	100	35.47	+5
			TYPE III				
2	6.3×10^6	35.71	0.0	2	6.3×10^6	36.00	0.0
100	100	35.65	+2	100	98	35.94	0.0
200		35.67	0.0	200		35.92	-1
300		35.67	-3	300		35.87	+4
400	103	35.61	-3	400	103	35.89	0.0

400	103	35.61	-3	400	103	35.89	0.0

Table 9a. Dynamic modulus of elasticity, weight, and change in gauge length of 3" by 4" by 32" prestressed concrete beams after various periods of repetitive freezing and thawing (fast cycle) - Series III.

The first line in each tabulation under dynamic modulus shows initial modulus in millions of psi. All other values are in per cent of the initial modulus.

A (+) sign and (-) sign denote an increase and decrease, respectively, from the initial gauge length.

MOIST CURED				STEAM CURED			
No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)	No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)
			TYPE I (zero psi stress)				
10	6.2×10^6	34.84	+0.0	10	6.4×10^6	34.82	+0.0
10	100	34.84	+0.0	10	100	34.82	+0.0
50	100	34.88	+1.0	50	100	34.87	+0.0
100	100	34.89	+4.0	100	100	34.87	+0.0
150	102	34.91	+9.0	150	100	34.90	+1.0
200	102	34.94	+5.0	200	100	34.91	+2.0
250	102	34.95		250	100	34.94	+1.0
300	102	34.97	+1.0	300	100	34.93	+0.0
350	100	34.94	+4.0	350	98	34.93	-2.0
400	100	34.98	+6.0	400	100	34.93	+0.0
			TYPE III				
10	5.8×10^6	34.12	+0.0	10	6.3×10^6	34.30	+0.0
10	100	34.12	+0.0	10	100	34.30	+0.0
50	100	34.18	+0.0	50	100	34.33	+0.0
100	100	34.20	+0.0	100	100	34.33	+0.0
150	100	34.23	+0.0	150	100	34.34	+1.0
200	100	34.23		200	100	34.39	+4.0
250	102	34.27		250	98	34.39	+1.0
300	100	34.25	+3.0	300	98	34.39	+0.0
350	98	34.23	+3.0	350	98	34.39	+0.0
400	99	34.23	+4.0	400	97	34.41	+3.0

Table 9b. (cont.)

MOIST CURED				STEAM CURED			
No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)	No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)
			TYPE I (500 psi stress)				
10	6.1×10^6	35.08	+0.0	10	6.2×10^6	35.44	+0.0
100	100	35.08	+0.0	100	101	35.48	+0.0
200	102	35.15		200	102	35.52	
300	100	35.12	+3.0	300	101	35.51	+1.0
400	98	35.12	+13.0	400	97	35.53	+1.0
			TYPE III				
10	5.9×10^6	34.40	+0.0	10	5.7×10^6	34.51	+0.0
100	101	34.42	+0.0	100	102	34.53	+0.0
200	101	34.45		200	101	34.56	+8.0
300	100	34.47	+1.0	300	101	34.57	+4.0
400	100	34.48	-1.0	400	100	34.57	+0.0
			TYPE I (1000 psi stress)				
10	6.3×10^6	35.53	+0.0	10	6.4×10^6	35.53	+0.0
100	100	35.57	-2.0	100	101	35.53	+0.0
200	100	35.69	+12.0	200	101	35.56	
300	99	35.65	+4.0	300	100	35.57	-2.0
400	99	35.72	+6.0	400	99	35.95	-5.0
			TYPE III				
10	5.9×10^6	34.50	+0.0	10	5.6×10^6	33.96	+0.0
100	101	34.51	+6.0	100	101	33.97	+3.0
200	102	34.55	+15.0	200	101	34.00	+10.0
300	101	34.58	+16.0	300	101	34.05	
400	100	34.58	+16.0	400	99	33.96	

400	100	34.58	+16.0	400	99	33.96	

Table 9 c. (concl.)

MOIST CURED				STEAM CURED			
No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)	No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)
			TYPE I (1500 psi stress)				
10	6.1×10^6	35.23	+0.0	10	6.3×10^6	35.19	+0.0
100	103	35.22	+0.0	100	102	35.20	+0.0
200	103	35.33		200	102	35.30	+6.0
300	102	35.30	-1.0	300	102	35.27	-5.0
400	101	35.32	-3.0	400	101	35.29	+3.0
			TYPE III				
10	5.8×10^6	35.07	+0.0	10	5.6×10^6	34.13	+0.0
100	102	35.10	+0.0	100	101	34.16	+1.0
200	102	35.17		200	101	34.12	-1.0
300	102	35.11		300	101	34.14	-10.0
400	100	35.20	+0.0	400	101	34.14	+5.0
			TYPE I (2000 psi stress)				
10	6.2×10^6	35.38	+0.0	10	6.4×10^6	35.31	+0.0
100	100	35.42	+2.0	100	100	35.32	-2.0
200	100	35.44		200	100	35.39	
300	100	35.46	-1.0	300	100	35.35	+0.0
400	100	35.46	+0.0	400	99	35.43	+9.0
			TYPE III				
10	6.0×10^6	34.81	+0.0	10	5.6×10^6	33.98	+0.0
100	101	34.87	+6.0	100	100	34.00	+2.0
200	101	34.90	+4.0	200	100	34.00	+0.0
300	101	34.91	+2.0	300	100	34.00	-3.0
400	100	34.93	+4.0	400	98	34.00	+0.0

Table 10a. Dynamic modulus of elasticity, weight, and change in gauge length of 3" by 4" by 32" prestressed concrete beams after various periods of repetitive freezing and thawing (fast cycle) - Special Series.

The fourth line in each tabulation under dynamic modulus shows initial modulus in millions of psi. All other values are in per cent of the initial modulus.

A (+) sign and (-) sign denote an increase and decrease, respectively, from the initial gauge length.

MOIST CURED				STEAM CURED			
No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)	No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)
SERIES II			TYPE I (zero psi stress)				
400				400	99	34.58	+6.0
400(dry)				400(dry)	87	34.32	+0.0
400(wet)				400(wet)	102	34.78	+10.0
405				405	6.0×10^6	34.77	+0.0
450				450	100	34.71	+3.0
500				500	100	34.77	+1.0
550				550	100	34.72	+2.0
600				600	100	34.72	+2.0
700				700	100	34.70	+4.0
800				800	100	34.70	+6.0
			TYPE III				
400	95	35.15	+6.0	400	100	35.51	+2.0
400(dry)	61	34.84	-2.0	400(dry)	75	35.17	-8.0
400(wet)	107	35.30	+10.0	400(wet)	105	35.68	+11.0
405	5.7×10^6	35.28	+0.0	405	6.0×10^6	35.66	+0.0
450	98	35.25	+4	450	101	35.61	+0.0
500	98	35.25	+0	500	101	35.66	+2.0
550	96	35.25	+2	550	101	35.65	+2.0
600	96	35.24	+2	600	101	35.63	+4.0
700	96	35.25	+4	700	103	35.60	+0.0
800	100	35.26	+6	800	105	35.58	+7.0

Table 10a. (cont.)

MOIST CURED				STEAM CURED			
No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)	No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)
SERIES II			TYPE I (2000 psi stress)				
400	100	35.10	+0.0	400	100	35.47	+5.0
400(dry)	100	34.89	-4.0	400(dry)	100	35.21	-11.0
400(wet	100	35.28	+11.0	400(wet)	100	35.60	+12.0
405	6.2×10^6	35.28	+0.0	405	6.5×10^6	35.60	+0.0
500	100	35.25	+0.0	500	100	35.57	+0.0
600	100	35.20	+1.0	600	100	35.57	+1.0
700	100	35.20	+2.0	700	100	35.54	+1.0
800	100	35.17	+3.0	800	100	35.51	+2.0
			TYPE III				
400	100	35.61	--3.0	400	100	35.89	+0.0
400(dry)	100	35.32	-18.0	400(dry)	100	35.69	+2.0
400(wet)	99	35.77	+18.0	400(wet)	100	36.02	+12.0
405	6.5×10^6	35.77	+0.0	405	6.5×10^6	36.03	+0.0
500	100	35.75	+0.0	500	100	36.00	+0.0
600	100	35.72	+0.0	600	100	35.97	+2.0
700	100	35.72	+2.0	700	100	35.97	+3.0
800	100	35.70	+4.0	800	100	35.96	+5.0

Table 10b. SERIES III

MOIST CURED				STEAM CURED			
No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)	No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)
			TYPE I (zero psi stress)				
400	98	34.98	+6.0	400	100	34.93	+0.0
400(dry)	100	34.79	-2.0	400(dry)	99	34.79	-1.0
400(wet)	100	35.07	+8.0	400(wet)	100	35.04	+8.0
405	6.4×10^6	35.06	+0.0	405	6.4×10^6	35.04	+0.0
500	100	35.09	+0.0	500	100	35.06	+1.0
600	100	35.03	+0.0	600	100	35.03	+2.0
700	100	35.04	+8.0	700	100	35.07	+8.0
800	100	35.00	+4.0	800	100	35.02	+7.0
			TYPE III				
400	100	34.23	+4.0	400	100	34.41	+3.0
400(dry)	97	34.01	-12.0	400(dry)	100	34.25	-7.0
400(wet)	98	34.31	+8.0	400(wet)	100	34.46	+5.0
405	5.8×10^6	34.32	0.0	405	6.1×10^6	34.49	+0.0
500	100	34.33	+3.0	500	102	34.46	+0.0
600	100	34.29	+6.0	600	102	34.44	-1.0
700	100	34.29		700	100	34.45	+4.0
800	100	34.27	+9.0	800	102	34.40	+1.0

Table 10-b. (cont.)

MOIST CURED				STEAM CURED			
No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)	No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)
			TYPE I (1000 psi stress)				
400	100	35.72	+6.0	400	99	35.95	-5.0
400(dry)	100	35.52	+4.0	400(dry)	99	35.40	-2.0
400(wet)	100	35.80	+10.0	400(wet)	99	35.70	+9.0
405	6.3×10^6	35.78	+0.0	405	6.4×10^6	35.69	+0.0
500	100	35.80	+0.0	500	100	35.71	-1.0
600	100	35.75	+2.0	600	100	35.68	-1.0
700	100	35.77		700	100	35.68	+3.0
800	100	35.74	+3.0	800	100	35.60	-1.0
			TYPE III				
400	100	34.58	+16.0	400	100	33.96	
400(dry)	100	34.43	+0.0	400(dry)	100	33.86	0.0
400(wet)	100	34.74	+8.0	400(wet)	100	34.16	+8.0
405	5.9×10^6	34.74	+0.0	405	5.5×10^6	34.16	+0.0
500	102	34.75	-1.0	500	100	34.15	-2.0
600	102	34.70	-1.0	600	100	34.10	-3.0
700	102	34.71	+4.0	700	100	34.12	
800	100	34.70	+0.0	800	100	34.08	-2.0

Table 10-b. (concl.)

MOIST CURED				STEAM CURED			
No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length (10^{-4} in.)	No. of cycles of test	Dynamic modulus	Weight (lb)	Change in 10" gauge length
TYPE I (2000 psi stress)							
400	98	35.46	+0.0	400	98	35.43	+9.0
400(dry)	98	35.33	+4.0	400(dry)	100	35.25	+2.0
400(wet)	98	35.60	+12.0	400(wet)	98	35.51	+11.0
405	6.3×10^6	35.59	+0.0	405	6.4×10^6	35.53	+0.0
500	100	35.58	+0.0	500	100	35.53	+0.0
600	100	35.55	+1.0	600	100	35.49	+1.0
700	100	35.57	+1.0	700	100	35.52	+3.0
800	100	35.54	+2.0	800	100	35.51	+3.0
TYPE III							
400	100	34.93	+4.0	400	100	34.00	+0.0
400(dry)	100	34.69	+3.0	400(dry)	100	33.85	-4.0
400(wet)	98	35.08	+11.0	400(wet)	98	34.22	+9.0
405	6.0×10^6	35.08	+0.0	405	5.5×10^6	34.20	+0.0
500	100	35.08	+0.0	500	100	34.18	+0.0
600	100	35.03	+2.0	600	100	34.16	-1.0
700	100	35.03	+4.0	700	100	34.20	+3.0
800	100	35.03	+4.0	800	100	34.17	-1.0

PCI STANDARD BUILDING CODE FOR PRESTRESSED CONCRETE PRESENTED FOR REVIEW

Copies of our tentative Building Code have been distributed to our membership and to registrants at the PCI Fifth Annual Convention. Constructive criticism is now invited. It is suggested that all discussion and criticism be made clear and concise and referred to the respective sections of the Code. They should be sent to the Committee Chairman, Professor T. Y. Lin, with minimum of two duplicates, to his address listed below. Seven copies are preferred if convenient. Closing date for their acceptance is March 1, 1960. Copies have also been sent to members of the Federation Internationale de la Precontrainte for their review and comment.

The committee intends to revise and publish the Code in its final form, based on constructive criticism received.

Correspondence should be addressed to Professor T. Y. Lin, Chairman PCI Standard Building Code Committee, Engineering Materials Library, University of California, Berkeley 4, California.

A questionnaire pertaining to air-entrained concrete mailed to some members by Assistant Executive Secretary Norman Scott brought the following significant replies:

State	Does your State specify Air-entrained Concrete? All work or just some of it?	Do you know why they use it . . . or don't use it?	What air content is specified?
OKLAHOMA	No	No	None
IOWA	No	Not considered necessary for structural units. Air required in deck slabs over p/c beams.	4 to 6%
MICHIGAN	Yes. All of it.	Improved durability against freezing and thawing conditions and salt.	4 to 5½%
FLORIDA	Yes	Improved durability with respect to salt action.	3 to 4%
TEXAS	Texas does not normally specify air-entrained concrete. They do allow air-entrained as an additive to improve workability.	Probably don't use it because freezing and snow removal are very small problems in Texas.	None
OREGON	Oregon specifies air-entrained concrete for part of their prestressed work.	They use air-entrained concrete for prestressed members that are going to be used in an area where weather conditions are such that freezing and thawing occur severely.	4 to 6%
N. J.	No		
COLORADO	No, but it does not prohibit its use		Not to exceed 3%

The TAC conducted a survey recently among Active Members to obtain their views and experience on concrete release strengths. A questionnaire was mailed that brought many interesting replies, some of which are published below. (Editor.)

Name of Company Producer	Recommended Concrete Strength at Release 3/8" str.	7/16" str.	1/2" str.	Comments
Dura-Stress	4,000	4,000	4,000	Because stresses at transfer usually run at around 2,400 psi (after elastic shortening) and this is $0.60f'_{ci}$. We feel that $0.60\ f'_{ci}$ is a good limitation for bridge products. Do not believe bond will be any problem on ½" strand.
Midwest Prestressed Concrete Co.	3,500	4,000		3,500 — We produced beams with this release strength for 5 years without any visual or measured failure. 4,000 for 7/16"—this is all we have ever used.
Missouri Pres-Crete	3,500	3,500 or 4,000	4,000	3/8" — 3,500 especially for low design stresses (less than 800 psi) — No slippage observed on any projects we had for the last 3 years. All projects appear satisfactory. 7/16" — 3,500 for low design stresses (less than 800) 4,000 above 800 psi. No slippage observed on any project manufactured so far. 1/2" — We believe that this value will be satisfactory. We have had no experience with 1/2" strand. Judging from experience on D.T.'s, beams and girders for buildings the 4,000 psi release stress should be satisfactory.
N. C. Products Corp.	3,500	3,500		The cylinder strength at release should be decreased (3,500 psi min.) or increased to suit the stresses induced in the concrete at the time of release. The size of strand (excluding 1/2" which we have not yet used) should not govern release strength. The only effect of the strand size would be the local stresses at the ends of the member due to "swelling" of the strand. We have found 3,500 psi concrete suitable for both 3/8" and 7/16" strand in this respect. (f'_{ci} = 3,500 psi.)
Grenada Concrete Prod.	4,000	4,000		3/8" — Our experience has proven this value to be adequate. 7/16" — No higher than 4,000 psi. Our experience has proven this value to be adequate.
Material Service Corp.	3,500	4,000	4,000	Have developed unnecessary cracks when stress released at lower level.
Penna. Prestress, Inc.	4,000	4,500		Because we have had no release problem to date nor excessive cambers.

Name of Company Producer	Recommended Concrete Strength at Release 3/8″ str.	7/16″ str.	1/2″ str.	Comments
Edw. Campbell Co.	3,500 to 4,000			Depending on stresses in section at release.
Utah Sand & Gravel	3,500	4,000	4,000	3/8″. — Unplanned releases at 2,500 have shown no ill effects, 3,500 provides adequate margin of safety. 7/16″ & 1/2″ — Values of these arbitrary until we have sufficient experience to justify a change. We feel, however, from available information above values are adequate.
Ballard-Rice Prestressed	3,500			3,500 psi minimum under all conditions. Higher strengths might be necessary under some designs. Each design is an individual problem.
Lamar Pipe & Tile	4,000	4,000		3/8″ & 7/16″ — 4,000 psi safe and adequate, as we never have had trouble on this basis. Below 4,000, the allowable stresses, using current ACI recommendations, would become too low, thereby causing overdesign of members. A higher figure would be a stupid and unjustifiable spec., as a strength greater than 4,000 psi will accomplish nothing but hurt production.
Arnold Stone Co., Inc.	3,500	3,500		Our experience has shown that a cylinder strength of 3,500 p.s.i. for 5,000# concrete is sufficient for all strands. We use 3/8 and 7/16 strands.
Dickerson Structural Concrete	4,000	4,000	4,000	3/8″ & 7/16″ — Experience shows this strength to be adequate based on allowable stresses at release of 2,400 psi comp. & 480 psi tension also — no bond failure has been evident. 1/2″ — We feel that the 4000 psi release strength is so conservative (or $f'_{ci} = \frac{\text{initial prestress}}{0.6}$ if specific design requires) that it would be adequate for 1/2″ strand. We do not have experience with 1/2″ strand to comment on bonding.
Prestressed Concrete Products, Inc.	4,000	4,000		Camber, bond, etc. appear satisfactory and 4,000 psi release is still economical.
Atlas Structural Concrete, Inc.	4,000	4,000	4,000	3/8″ & 7/16″ — Proven to be satisfactory by several years experience. 1/2″ — In all our bridge work the use of a 4,000 psi release strength has proven beyond question to give excellent results. Good structural behavior of these members in continuous use has borne this out conclusively.
Nashville Breeko Block Co.	3,500	4,000	4,000	Adequate

Name of Company Producer	Recommended Concrete Strength at Release 3/8″ str.	7/16″ str.	1/2″ str.	Comments
Macon Prestressed Concrete Co.	4,000	4,000	4,000-4,500	3/8″ & 7/16″ — Good results from past experience. 1/2″ — possibly the 1/2″ should be increased. Also possibly the 3/8″ should be reduced slightly. This should be verified by test.
Capitol Prestress Co.	4,000	4,000	4,000	See no reason to change present criteria since positive bond is developed and 4,000 psi cylinder strength gives ample safety for the concrete.
Florida Prestressed Concrete Co., Inc.	3,600	4,000	4,500	3/8″ — Based on 5,000 psi concrete, tests using lower strengths at release did not prove satisfactory and very little, if any, benefit was realized with a higher value. 7/6″ — Same as above. 1/2″ — No experience. I state this figure just as an off-hand observation taking in consideration the higher bond stresses required with the 1/2″ strand.
West Coast Shell Corporation	3,600	4,500	4,500	3/8″ — No appreciable gain after attaining 3,600# — taking time into consideration. 7/16″ — More bond required. 1/2″ — No experience, but estimate 4,500 because more bond stress required.
Strangbetong Stockholm				7/16″ — Not less than 5,000 lb/in^2 on cubes or 5,000/1.25 = 4,000 cylinders. (Important note: Europeans measure cubes in lieu of cylinders. Conversion factor: approx. 1.25 5,000 cube = $\frac{5{,}000}{1.25}$ = 4,000 cyl. C. C. Zollman.) The transmission length necessary depends to a large extent on the size of the product — the smallest sections being decisive. The cylinder strength at release must also be regarded as a general level of quality with regard to creep in the concrete. We do not think the size of strand ought to decide the minimum strength at release.
Wilson Concrete Co.	3,500-4,500	3,500-4,500		3/8″ & 7/16″ — with these values ample strength has been developed for bond and camber control if properly designed.
F-S Prestress, Inc.	4,000	4,000		It is a value which is workable production-wise and at the same time assures a satisfactory level of strength.
Virginia Prestressed Concrete	4,000	4,000	4,000-4,200	3/8″ & 7/16″ — Satisfactory. 1/2″ — No experience — bonding questionable except for higher strengths.

Name of Company Producer	Recommended Concrete Strength at Release 3/8″ str.	7/16″ str.	1/2″ str.	Comments
Lewis Mfg. Co.	4,000	4,000	4,000	3/8″ & 7/16″ — More uniform camber. Less spalling and cracking in handling. 1/2″ — No experience. Since the most economical amount of prestress usually is around a final value of P/A = 1,000 psi, the initial P/A is nearly the same regardless of what size strands are used.
Crider & Shockey, Inc.	3,500	3,500	3,500	3/8″ — Experience releasing at this strength shows no ill effects in the concrete. However, according to the joint committee recommendations for the allowable concrete stress at time of release, releasing at this stress will sometimes require non-prestressing tensile reinforcement so that the code limitations will not be exceeded. In the lighter structural members this becomes quite expensive, and it is actually more economical to attain the higher release strength. Since performance records are satisfactory, the indicated stress is recommended. This comment also for 7/16″. 1/2″ — Not presently using. As previously stated, however, the strand spacing may have to be increased so that the above recommended stresses in the concrete will not be exceeded.
Ben C. Gerwick, Inc.	3,500	4,000	4,000-4,500	3/8″ — Proven adequate. 7/16″ — Bond. 1/2″ — Not using — don't know. Suggest 4,000-4,500 — reason: bond.
Basalt Rock Co., Inc.	3,000	3,500		Structurally sound based on actual tests conducted by Basalt Rock Co.
Delta Prestress Concrete, Inc.	3,000	3,500	4,000	3/8″ — This value has been used enough successfully to show its practicality. 7/16″ — No experience. 1/2″ — No experience. We have used 3,000 psi release on building work including State inspected schools and find it quite adequate. The other values are in proportion to the bond stress assuming that the bond strength of the concrete is proportional to the cylinder strength of release. Note: Higher transfer stresses are very often useful in obtaining maximum use of concrete. f'_{ci} up to 4,000 psi is OK and is recommended if necessary to keep $f'_c \leq 0.60\ f'_{ci}$
Superior Sand & Gravel		3,000		3,000 or higher as required by $f'_c = 0.60\ f'_{ci}$. 3 years of satisfactory experience with 3,000 psi release in building members. No experience with other strands.

Name of Company Producer	Recommended Concrete Strength at Release 3/8″ str.	7/16″ str.	1/2″ str.	Comments
Appalachian Consulting Engineers, Inc.	3,500	3,500	4,000	3/8″ — Assuming design is adequate and bond stress only controls. No indication of spalling or excessive bond stresses. 7/16″ — Same. 1/2″ — The plant has had no actual experience with 1/2″ strand but feel should be more conservative.
Concreto S.A.	3,500			In the beams we made we didn't have any problems with it. We didn't have one cracked girder.
Thomas Concrete Pipe Co.				No experience with 1/2″ strand but recommend for all: 4,000 psi for f'_c = 5,000 psi and 4,800 psi for f'_c = 6,000 psi. Our experience indicates that release strength should be approximately 0.8 f'_c in order to control camber and to properly develop the necessary bond between strand and concrete.
Rocky Mountain Prestress, Inc.	3,500	3,500	4,000	Bond — bending stresses — camber
C. W. Blakeslee & Sons, Inc.	3,500	3,500	3,500	In general, 3,500 psi transfer strengths sufficient to transfer bond — occasionally we use higher values in order to minimize concrete creep which increases rapidly at high stress values.
Duval Engineering & Contr. Co.				
Concrete Materials, Inc.	3,500	3,500	3,500	3/8″ — Allows good E value for behavior. 7/16″ — Have used over two year period in building girders and beams with no objectionable behavior. Perfectly safe to use. 1/2″ — See no difference than with 7/16″. Should be same for these size ranges. We have produced over $6 million worth of prestressed work using a release value of 3,500 and have never had the first instance of trouble and don't expect any.
Southeastern Concrete Prod. Co.	3,500	3,500	3,500	3/8″ & 7/16″ — If the compression induced in the concrete at release is under 1,000 psi. 1/2″ — Have not used. During 1955 and 1956 we used the value of 3,500 on our prestressed products and it was very satisfactory. 4,000 psi quite often imposes a hardship on the manufacturer as this high strength is difficult to obtain in a short period of time and delays removing the product from the casting bed. We feel that 3,500 psi is adequate and that the extra expense of obtaining strength above this value is unnecessary.
Precast Industries	3,500			

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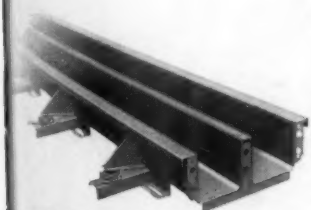

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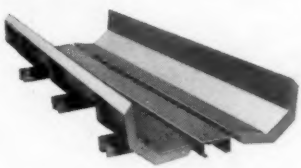

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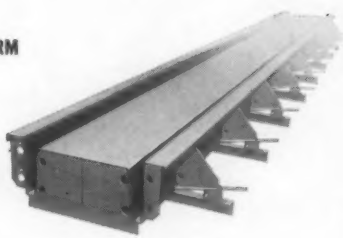

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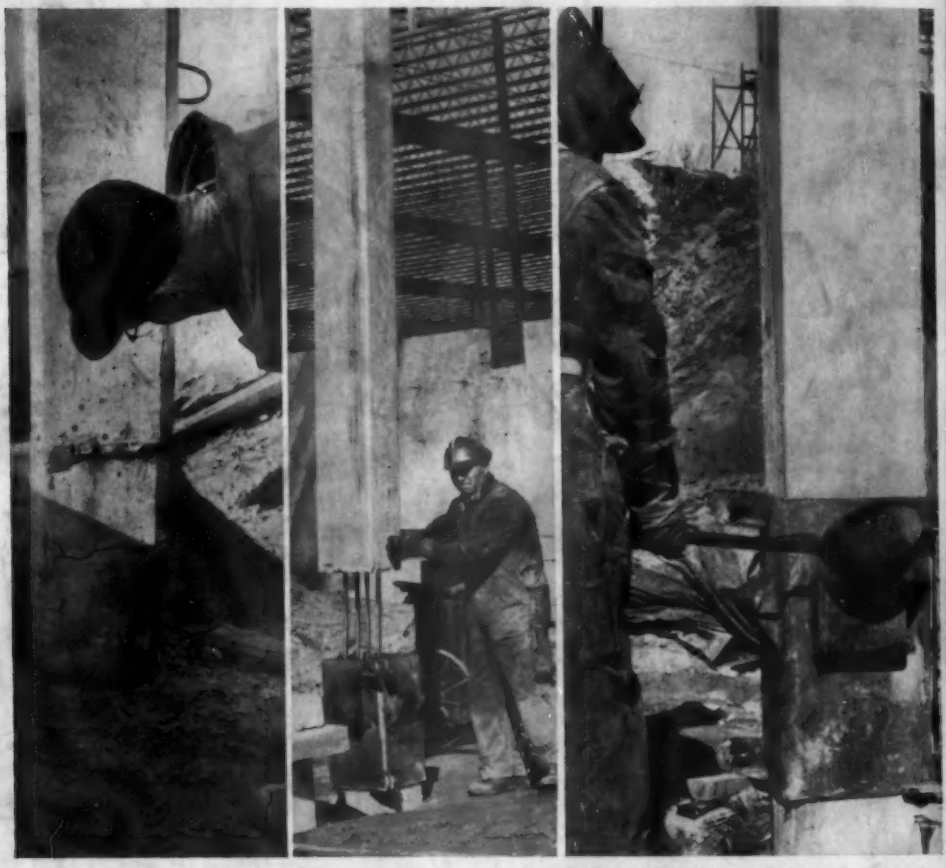

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